FOUNDATION MODEL FOR REINFORCED GRANULAR FILL - SOFT SOIL SYSTEM AND ITS SETTLEMENT RESPONSE

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CERTIFICATE

It is certified that the work contained in the thesis entitled "**FOUNDATION MODEL FOR REINFORCED GRANULAR FILL - SOFT SOIL SYSTEM AND ITS SETTLEMENT RESPONSE**" by **SANJAY KUMAR SHUKLA**, has been carried out under my supervision and that this work has not been submitted elsewhere for a degree.

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The thesis pertains to the development of a mechanical foundation model for representing the behaviour of geosynthetic-reinforced granular fill - soft soil systems which are used as foundations for shallow footings, embankments, storage tanks, temporary working platforms, car parks, paved and unpaved roads etc. in many parts of the world. Each sub-system of the reinforced soft soil system is idealised by the mechanical foundation model elements, such as rough elastic membrane, Pasternak shear layer, Winkler springs, and dashpots which are commonly adopted for solving many soil-foundation-structure interaction problems in geotechnical engineering. The suggested foundation model incorporates various aspects of the behaviour of the geosynthetic-reinforced granular fill - soft soil system, consideration of which may be necessary in many field situations. These aspects include the horizontal stresses induced in the granular fill, the compressibility of the granular fill, and the time-dependent behaviour of the soft subgrade. To study the effect of prestressing the geosynthetic reinforcement on the settlement

characteristics of the reinforced soil system, a foundation model element using a *stretched rough elastic membrane* is developed which is incorporated in the proposed foundation model.

The equations governing the response of the proposed foundation model are derived with the general assumptions that the geosynthetic reinforcement is linearly elastic, rough enough to prevent slippage at the interface with soil and has no shear resistance. A rigid-perfectly plastic friction model is adopted to represent the behaviour of the fill-geosynthetic interface in shear. The modulus of subgrade reaction is assumed to have a constant average value with depth of the foundation soil and also with time. The equilibrium of different elements of the geosynthetic-reinforced granular fill - soft soil system for two different cases: (i) plane strain problems, and (ii) axi-symmetric problems are considered separately to obtain the settlement response of the proposed foundation model.

The numerical solutions are obtained by an iterative finite difference scheme and the results are presented in a nondimensional form. The parametric studies are carried out to bring out clearly the effects of various parameters of the foundation model on the settlement response. Wherever it is possible, the settlement predictions obtained by the foundation model are compared with those computed using the mechanical foundation models of similar nature, suggested in recent past.

Based on the present study, it is found that the proposed foundation model for the geosynthetic-reinforced granular fill - soft soil system is well suited to evaluate the settlement response over a large range of various parameters and may be used in situations of large deformations where most of the existing mechanical foundation models

for geosynthetic-reinforced soil are not applicable. Comparisons of the settlement predictions by the proposed foundation model with those computed using the existing mechanical foundation models show similar trend of results for common model parameters.

As expected, the horizontal stresses induced in the granular fill in a geosynthetic-reinforced granular fill - soft soil system result in settlement reductions throughout the reinforced zone. Prestressing the geosynthetic reinforcement in geosynthetic reinforced granular fill - soft soil systems is found to be very effective in reducing both the total and the differential settlements of the loaded region.

It is observed that the compressibility of the granular fill has an appreciable influence on the settlement response of the geosynthetic-reinforced granular fill - soft soil systems as long as the stiffness of the granular fill is less than an optimum value. Using the proposed foundation model, it has become easily possible to evaluate the settlement response of the geosynthetic-reinforced granular fill - soft soil systems at any stage of one-dimensional consolidation of the soft foundation soil in saturated conditions. The numerical approach, to solve the governing equations of the model response, is found to be very efficient in terms of economy of computations and takes only few seconds of CPU time to obtain the settlements within the reinforced zone.

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NOTATIONS

a	radius of loaded region
b	radius of reinforced zone
B	half width of strip loading
C_v	Coefficient of consolidation
G_t	Shear modulus of upper shear layer
G_t^*	Nondimensional shear parameter for upper shear layer (= $G_t H_t / k_s B^2$ in plane strain case, and = $G_t H_t / k_s a^2$ in axi-symmetric case)
G_b	Shear modulus of lower shear layer
G_b^*	Nondimensional shear parameter for lower shear layer (= $G_b H_b / k_s B^2$ in plane strain case, and = $G_b H_b / k_s a^2$ in axi-symmetric case)
H_t	Thickness of upper shear layer
H_b	Thickness of lower shear layer
H_s	Thickness of soft soil layer
i	Space subscript
j	Time subscript
k_f	Modulus of subgrade reaction for granular fill
k_s	Modulus of subgrade reaction for soft foundation soil
K	Lateral stress ratio
L	Half width of reinforced zone
q	Load intensity
q^*	Nondimensional load intensity (= $q/k_s a$)
q_0	Uniform load intensity
q_t	Vertical stress at the top of membrane
q_b	Vertical stress at the bottom of membrane
q_0^*	Nondimensional uniform load intensity (= $q_0/k_s B$ in plane strain case, and = $q_0/k_s a$ in axi-symmetric case)
r	Radius vector of polar co-ordinates
R	Nondimensional radial distance (= r/a)

R_c	Overconsolidation ratio for the granular fill
T	Mobilised tensile force in membrane per unit length
T^*	Nondimensional mobilised tensile force per unit length (= T/k_sB^2 in plane strain case and = T/k_sa^2 in axi-symmetric case)
T_p	Pretension in the membrane per unit length
T_p^*	Nondimensional mobilised tensile force per unit length (= T_p/k_sB^2 in plane strain case and = T_p/k_sa^2 in axi-symmetric case)
T_v	Time factor
U	Degree of consolidation for the soft foundation soil
w	Vertical surface displacement
W	Nondimensional vertical surface displacement (= w/B for plane strain case, and = w/a for axi-symmetric case)
x	Distance from the centre of loaded region
X	Nondimensional distance (= x/B)
X_k	Dimensionless variable parameters ($k = 1, 2, 3, 4$)
α	Modular ratio (= k_f/k_s)
θ	Slope of membrane
μ_t	Interfacial friction coefficient at the top face of membrane
μ_b	Interfacial friction coefficient at the bottom face of membrane
τ_{xz}	Shear stresses on vertical face of the shear layer in plane strain case
τ_{rz}	Shear stresses on vertical face of the shear layer in axi-symmetric case
ϕ	vectorial angle of polar co-ordinates
ϕ'	Effective angle of shearing resistance

ϕ_R – (p. 9)

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Fine-grained saturated soils exist near most river estuaries and coastal areas around the world. Incidentally, these are often the areas where industrialisation is the heaviest. In these areas the poor soil conditions often create problems to geotechnical engineers associated with the foundation design and construction works of civil engineering structures such as buildings, highways, railways, airfields, embankments, dams, storage tanks, car parks and temporary working platforms. Because of low bearing capacity, such soils are generally not able to withstand heavier structures. In several situations, the bearing capacity of foundation soils is low (less than 7.0 kPa) and they would hardly support the weight of an individual. Sometimes, even if the structural failure is avoided the excessive settlements caused to the structures adversely affect their functional utility. There are many existing stable structures resting on soft foundation soils wherein the excessive settlements have adversely affected their functional utility.

Some of the traditional options to solve the problem are: change of site, designing the proposed structure accordingly, excavation and replacement with suitable soil, deep

foundations placed through the unsuitable soils, wait until natural consolidation occurs, or stabilisation with injected additives. In many situations, the expensive and time-consuming conventional designs and several environmental constraints such as the lack of good construction sites and non-availability of good-quality granular materials in required quantities led to the adoption of modern techniques of *ground improvement* which is a viable solution, both technically and economically.

The history of civil engineering is closely connected with foundation work on soils of poor bearing capacity and with modification of these soft soils. Looking for ever more effective and more economical methods of founding engineering objects, quite a number of soil improvement techniques have been developed in past, from the classical soil replacement and preliminary loading to modern solutions, such as precompression (by preloading or by dewatering along with or without the use of sand drains/wick drains) compaction by means of explosions, vibroflotation, dynamic consolidation, compaction grouting, compaction with water jets, thermal stabilisation, ground freezing, chemical stabilisation, use of stone columns/granular piles or lime piles, electrokinetic stabilisation and soil reinforcement. Among all these existing methods of ground improvement, only ground freezing is applicable to all types of soil, provided it is moist. Other methods are most suitable for particular groups of soils, usually either cohesive or cohesionless soils. The feasibility of a particular method is strongly related to the type of problem in hand, whereas the desirability of a particular method is largely perceived in terms of environmental impact and energy consumption.

Soil reinforcement has become a major part of geotechnical engineering practice over the last 30 years, and its use is growing rapidly as world-wide development poses an

increasing demand for land reclamation and the utilisation of soft foundation soils. However, the concept of soil reinforcement is not exactly new. The basic principles involved in soil reinforcement techniques are simple to grasp and have been used by human beings for centuries. In many countries, the use of soil reinforcement is common in village constructions since ancient times. Rope fibres and bamboo sheets are used to strengthen rural road bases and the foundation soil below low-cost and low-rise buildings. Vertically arranged rectangular grids of bamboo sheets and stalks of palm branches are used as the central core of mud walls.

The modern concept of soil reinforcement was proposed by Casagrande who idealised the problems in the form of a weak soil reinforced with high-strength membranes laid horizontally in layers (Westergaard, 1938). The modern form of soil reinforcement was introduced by Henry Vidal, a French architect and an engineer in the 1960s. Vidal's concept was for a composite material formed from flat reinforcing strips laid horizontally in a frictional soil, the interaction between the soil and the reinforcing members being solely by friction generated by gravity. This material was described by him as 'Reinforced Earth'. In his early work, Vidal regarded Reinforced Earth as a new kind of construction material, created by the association of particulate medium with reinforcement: This assembly formed a "volume which has cohesion". The basic attributes of soil reinforcement which are of particular advantage in geotechnical engineering are reductions in costs and ease of construction coupled with a basic simplicity which provides an attraction to engineers. The first major retaining walls using the Vidal concept were built near Menton in South of France in 1968, although Vidal has built structures earlier, starting in 1964.

Since the early development of reinforced earth technology in which mainly steel reinforcement (having corrosion problem) was used, alternative reinforcement materials have been investigated, ranging from stainless steel, aluminium, and fibreglass to nylon, polyester, polyamides, and other synthetics in the form of strips, meshes, and sheets. The most promising and exciting new materials belonging to the family of geosynthetics: geotextiles, geogrids, and geocomposites, were not regularised before 1973 when a geotextile reinforcement was applied in a bridge construction in Sweden. These are constructional materials with unique properties which, to a great extent, explain their dramatic increase in their use in so short a time span. Probably, no other specific class of items in civil engineering and related construction activities has had such a dynamic development.

In present-day geotechnical engineering, the geosynthetics are utilised to serve five major functions: separation, reinforcement, filtration, drainage, and moisture barrier. Reinforcement is one such function that is usually available either as a primary or as a secondary function. A large number of model tests carried out almost throughout the world reveal that most soft soils exhibit improved behaviour with respect to their load-settlement response whenever a geosynthetic is provided usually along with the granular fill.

The geosynthetic-reinforced granular fill - soft soil systems are now being used very frequently as foundations for unpaved roads, shallow footings, low embankments, oil drilling platforms, heavy industrial equipment, car parks, and closure covers for tailing dams etc. Such reinforced soil systems provide improved bearing capacity and reduced settlements by distributing the imposed loads over a wider area of weak subsoil. In conventional construction technique without the use of any reinforcement, a thick granular

layer is needed which may be costly or may not be possible specially in the sites of limited availability of good-quality granular materials. Moreover, the simplicity of the basic principles and the economic benefits over the conventional approaches make it very attractive to the designers. Also, use of soil reinforcement provides numerous other indirect benefits, such as speedy construction time, ease in construction, graceful appearances, etc.

1.2 SCOPE AND ORGANISATION OF THE PRESENT WORK

A critical review of available literature pertaining to the near-surface reinforced foundation soil has been presented in Chapter 2. From the review of literature, it has been observed that there are various aspects like vertical shear stress transfer at the fill-geosynthetic interface under large deformations, the prestressing of the geosynthetic reinforcement, the compressibility of the granular fill, and the time-dependent behaviour resulting from consolidation of the soft foundation soil, which need their considerations while using the geosynthetic-reinforced granular fill - soft soil system in the field and estimating its settlement under the applied load.

Chapter 3 deals with the development of a foundation model incorporating various factors, as stated above, which govern the settlement behaviour of geosynthetic-reinforced granular fill - soft soil systems. The model also incorporates the common parameters like the shear parameter of the granular fill and the characteristics of the soil-geosynthetic interface. The approach used in the development of the foundation model is mechanical and hence it is very simple to understand and to work with. Such an approach is being used in

soil-foundation-structure problems in investigating the gross behaviour and highlighting the major parameters and characteristics of the soil without reinforcement.

Chapter 4 describes with the detailed derivation of the response function of the proposed foundation model in plane strain conditions by considering equilibrium of different elements of the reinforced soil system. A rigid perfectly plastic friction model is adopted to represent the soil-geosynthetic interface characteristics. The finite difference scheme is used for solving the differential equations governing the response of the model. A detailed parametric study is carried out to bring out clearly the effect of each individual parameter on the settlement behaviour. The results are presented in nondimensional form for practical applications over a wide range of parameters. The present results are compared with those computed using earlier foundation models reported in literature wherever it is possible.

Chapter 5 presents the detailed derivation of the response function of the proposed foundation model for axi-symmetric conditions. In this case also, a detailed parametric study is carried out and results are presented in nondimensional form for practical applications. The comparison with earlier models is also carried out.

The summary and conclusions of the present work have been presented in Chapter 6 along with the recommendations for further work.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

The analysis of the interaction between structural foundations and supporting reinforced foundation soil is of fundamental importance. The results of such analysis generally provide information which can be used in structural design of the foundation and in the analysis of stresses and deformations within each sub-system of the reinforced soil. This interest has generated several theoretical and experimental studies in the area of reinforced soil-foundation interaction.

Several concepts have been developed to explain the reinforcing mechanism of the reinforcement used in the soil. Several design methods have been presented with assessment of bearing capacity of the soft foundation soil below the aggregate layer for the estimation of the surface deformation under the applied load. A large number of model tests have been conducted to bring out the effects of various parameters on the load-carrying capacity and the settlement characteristics of the reinforced soft soil system using geosynthetics. Many researchers have carried out finite element analysis by considering different reinforcing mechanisms at the soil-reinforcement interface and

provided valuable results. This chapter presents a comprehensive review of the state of the art on such theoretical and experimental studies associated with near-surface reinforced foundation soil under different sections.

2.2 REINFORCING MECHANISM

Apparently, different concepts have been advanced to define the basic mechanism of reinforced soil. The effect of inclusion of relatively inextensible reinforcements (such as, metals, fibre-reinforced plastics etc. having an high modulus of deformation) in the soil can be explained using either an induced stresses concept or an induced deformations concept.

The induced stresses concept is related to an apparent cohesion (Schlosser and Vidal, 1969). The tensile strength of the reinforcements and friction at soil-reinforcement interfaces give an apparent cohesion to the reinforced soil system. At the same time, the friction mobilised at the soil-reinforcement interfaces causes a rotation of principal stresses in the soil and modifies the initial state of stresses.

Several experimental studies have been carried out in triaxial apparatus on sand samples reinforced by horizontal thin aluminium plates (Schlosser and Long, 1972), and by horizontal nets of fibre glass (Yang, 1972) uniformly spaced. Failure occurred either by excessive lateral deformations due to sliding of the sand on the reinforcements or by the breakage of the reinforcements. Schlosser and Long (1972) showed that the reinforcements give to the sand an anisotropic cohesion directly proportional to their resistance to tension. Accordingly they interpreted the strength envelope for reinforced

sand at failure by breakage of the reinforcements, as that of a cohesive frictional Mohr-Coulomb material. They also showed that under small axial deformation, there is a rapid mobilisation of the apparent cohesion induced by the reinforcements, while the mobilisation of the internal friction angle is practically not affected by the presence of reinforcements. It was further stated that the reinforced sand is actually at a K_0 (coefficient of lateral stress at rest) state of stresses. Hence, as the vertical stresses increase, the horizontal stresses increase in direct proportion. Yang (1972) hypothesised that the tensile stresses built up in the reinforcements were transferred to the soil through sliding friction and caused an increase in the confining pressure. Both, the anisotropic or apparent cohesion approach (Schlosser and Long, 1972) and confining pressure approach (Yang, 1972) are related to each other and either one can be used to analyse failure by reinforcement breakage.

Hausmann (1976) interpreted the effect of reinforcing the sand on its strength characteristics considering a global apparent friction angle (ϕ_R). He assumed that when failure is caused by slip between the sand and the reinforcements, the reinforcing effect can be expressed in terms of an increased apparent friction angle.

The induced deformations concept was presented by Basset and Last (1978). This concept considers that the mechanism of tensile reinforcement involves anisotropic restraint of the soil deformations in the direction of the reinforcements (unidirectional, in the case of parallel reinforcement). This effect results in a rotation of the principal directions of the deformations tensor. Basset and Last (1978) suggested that more can be learnt by analysis of the modifications to strain fields caused by reinforcements than by the study of forces and stresses. Analysis of strain fields also suggests the ideal reinforcing

pattern below a shallow footing. The ideal pattern has reinforcement placed horizontally below the footing, which becomes progressively more vertical farther from the footing, i.e., reinforcement should be placed in the direction of major principal strain. This fact was stated by Hausmann (1990) in terms of stress. He stated that the tensile reinforcement is most effective if placed in the major principal plane, in the direction of the minor principal stress, which in many practical geotechnical problems is horizontal.

If relatively extensible reinforcements, such as geosynthetics, are used in the same manner as the inextensible reinforcements, they will also inhibit the development of internal tensile strains in the soil and develop tensile stresses. However, the differences between the influence of relatively inextensible and extensible inclusions exist and are significant in terms of the load-settlement behaviour of the reinforced soil system (McGown *et al.*, 1978). It was suggested that the behaviour of the reinforced soil system using extensible reinforcements does not fall within the concepts presented by Vidal (1969) for reinforced earth and therefore, was termed as ply-soil by McGown and Andrawes (1977). The ply-soil, e.g. the geosynthetic-reinforced soil has greater extensibility and smaller losses of post peak strength compared to sand alone or reinforced earth.

At present, the role of geosynthetic reinforcement in improving the load-carrying capacity and settlement characteristics of the geosynthetic-reinforced foundation soils is regarded in five different ways. The first is the increase of the subgrade bearing capacity by changing the failure mode, i.e. geosynthetics tend to force a general, rather than a local failure. The second is the reduction of the maximum applied stress due to a redistribution of the applied surface load below the geosynthetics by providing restraint of the granular fill if embedded in it or by providing restraint of the granular fill and the soft

foundation soil, if placed at their interface (referred to as slab effect or *confinement effect* (Giroud et al., 1984; Madhav and Poorooshasb, 1989; Sellmeijer, 1990; Hausmann, 1990)). The third is the supplementary support due to *membrane effect*, i. e. the deformed geotextile provides an equivalent vertical support (Giroud and Noiray, 1981; Bourdeau, et al., 1982; Sellmeijer et al., 1982; Love et al., 1987; Madhav and Poorooshasb, 1988; Bourdeau, 1989; Sellmeijer, 1990; Hausmann, 1990). Geosynthetics (particularly, geotextiles, but perhaps also geogrids) improve the performance by acting as a separator between the soft soil and the granular fill. This influence is known as *separation effect* of reinforcement. Very recently, Nishida and Nishigata (1994) have studied the separation function of geotextiles. Use of geogrids has another benefit owing to the interlocking of the soil through the apertures of the grid membrane known as *anchoring effect* (Guido *et al.*, 1986).

2.3 ANALYTICAL WORKS

Harrison and Gerrard (1972) presented an elastic theory applicable to reinforced earth. In this theory, the reinforced earth was considered to consist of soft soil layers reinforced by closely spaced parallel layers of extremely thin and extremely stiff material. It was pointed out that the suggested method of calculating stresses developed in soft soil and reinforcement can be used for the reinforced earth structures described by Vidal (1969) for cases where the stress field is not simply proportional to depth, e.g., under rafts and foundations. By using the techniques described, any zone of local overstressing in either the soft soil or the reinforcements can be defined.

Binquet and Lee (1975a) presented an analytical method for designing a reinforced earth slab foundation to carry a specified strip load and defined the term bearing capacity ratio (BCR) as: $BCR = q/q_0$, where, q_0 is the average contact pressures of the footing on the unreinforced soil; and q is the average contact pressure of the footing on the reinforced soil, both measured at the same vertical settlement. Based on the observed model test behaviour, three modes of bearing capacity failure were described: (i) shear failure above uppermost layer of reinforcement, (ii) ties pullout, and (iii) ties break. These failure modes depend on the arrangement and strength of the reinforcing strips. The boundary between the downward moving and outward moving soil was assumed to be defined by the loci of points of maximum shear stress at every depth determined from elastic theory, e.g., the Boussinesq equations. For lack of definitive data, it was arbitrarily assumed that tie force per layer varies inversely with the number of layers in the foundation. The idealised approximate cost analyses for the final designs of a realistic design problem indicated that if corrosion of the reinforcing material can be neglected, an overall saving of up to 100% may be realised over the cost of a conventional foundation. However, if corrosion must be considered, the savings will be significantly reduced. It was pointed out that maximum advantages are likely to be found for short-term construction involving heavy loads over inferior foundation soil conditions.

Giroud and Noiray (1981) developed a method for the design of geosynthetic-reinforced granular fill - soft soil systems used as unpaved roads. The method presented considers the reinforcement action of geotextiles alone and does not consider other beneficial effects of geotextiles, such as separation, filtration, and drainage. In the suggested method, an allowable rut depth is chosen and making use of a load spread angle and certain geometric

assumptions, the approximate deformed shape of the reinforcement is determined. Assuming that the reinforcement is firmly anchored outside the loaded area, the strain in the reinforcement, and hence also the reinforcement tension, can be deduced from the geometric changes. The reinforcement under the loaded area acts as a curved tensioned membrane, and this results in a higher normal stress on the upper surface of the reinforcement than on the lower surface, such effect of reinforcement was called *membrane effect*. The results were presented in the form of charts established using a combination of: (i) formulae relating aggregate thickness and traffic for unpaved roads without geotextile, and (ii) a quasi-static analysis comparing unpaved roads behaviour with and without geotextile. It was pointed out that the suggested method applies only to purely cohesive subgrade soils and is mostly applicable to roads subjected to light to medium traffic (1-10, 000 truck passengers over the lifetime of the road).

Bourdeau *et al.* (1982) presented a theoretical model for soil-membrane (geotextile) interaction using a probabilistic concept for the vertical stress diffusion in a particulate media (Harr, 1977). A generalisation was also offered for multilayered geotextile-reinforced systems. Using iterative numerical procedure, results were presented for a uniformly distributed load, and typical values of soil and geotextile properties. It was found that the efficiency of the geotextile increases as the subgrade underlying the membrane becomes more compressible, as long as the geotextile is sufficiently strong and possesses sufficient frictional resistance.

Ingold and Miller (1982) developed simple theories to model plane-strain compression of a reinforced clay cube, reinforced clay foundations and finally a reinforced clay wall. These theories are mainly based on one of the concepts of reinforcing mechanism that the

reinforcement embedded in the clay can be assumed to impart an equivalent undrained shear strength to the clay. A series of model tests were also described and test data presented was compared with the theories. It was pointed out that the comparisons, in general , showed sufficiently reasonable agreement.

Sawicki (1983) analysed reinforced earth considering it as macroscopically anisotropic and homogeneous composite. The gross behaviour was described based on a knowledge of the mechanical properties and interactive contribution of each component. The suggested rigid-plastic model for reinforced earth was applied for determining the lower bound estimate of the bearing capacity of a footing on reinforced earth. It was pointed out that the model does not consider one essential feature , i.e., a slippage between the soil and the reinforcement. It was also pointed out that both slippage and edge effects play an important role in the analysis of reinforced earth.

Broms (1987) described a method to stabilise very soft clay (mud) using woven geotextile and preloading. By constructing narrow berms on the geotextile, which is spread over the area to be stabilised, it is possible to place the fill required for preloading without exceeding the bearing capacity of the soil. The fabric should be stretched as much as possible before the stabilising berms are placed along the perimeter of the geofabric sheet in order to limit the penetration required to develop the necessary tension in the fabric. An analytical method was presented to evaluate the required strength of the fabric, as well as the width, height and spacing of the berms. It was reported that the method was used in Malaysia and Singapore with satisfactory results to stabilise very soft clay in settling ponds with shear strength of approximately 3 kPa so that the area could be used for construction. The method suggested that the maximum allowable strain in the fabric should be limited to

25% of the ultimate strain for permanent construction and to 50% for temporary structures.

Giroud *et al.* (1988) presented equations and charts to design soil layer-geosynthetic systems overlying voids such as cracks, sinkholes, and cavities. These equations and charts were developed by combining tensioned membrane theory (for the geosynthetic) with arching theory (for the soil layer), thereby providing a more realistic design approach than one that considers tensioned membrane theory only. It was suggested that granular soil should be well compacted to ensure arching because loose granular soils tend to contract when they are sheared or vibrated, which may destroy the arch. It should be noted that the equations and charts presented are essentially intended for granular soils; however, they can be used for saturated cohesive soils in the drained state, assuming that their cohesion is zero and provided that their drained friction angle is greater than 20^0.

Madhav and Poorooshasb (1988) proposed a 3-parameter mechanical model for geosynthetic-reinforced granular fill - soft soil system. In this model, the geosynthetic reinforcement was represented by a newly developed foundation model element using a rough membrane. The granular fill and the soft subgrade were idealised by Pasternak shear layer and a layer of Winkler springs respectively. The results at small displacement indicated the effect of granular fill to be more and significant than that of the membrane in reducing the settlements of the reinforced soft soil system. The effects of providing geosynthetic reinforcement were felt at higher loads or on soft soils.

Madhav and Poorooshasb (1989) investigated the effect of membrane in increasing the confining stress in the granular material with a consequent increase of the shear modulus (G) with distance with a Pasternak type foundation model. Analysis of a simple Pasternak

type foundation with variable G showed a significant reduction in the overall settlements as well as in the differential settlements due to the increased shear moduli. It was pointed out that extending the reinforcement beyond 2B (width of the footing) on either side of the centre of footing has less effect on settlements within the loaded region.

Bourdeau (1989) presented a model to assess tensile membrane action in a two layer soil system reinforced by a geotextile. The analysis was based on a two-dimensional plane strain model of the static equilibrium of an elastic membrane placed at the interface between a granular base and a compressible subgrade. The theory of stochastic stress diffusion in particulate media was used to describe the transmission of load through the gravel base, and a Winkler model was assumed for the subgrade. The model incorporates a Mohr-Coulomb criterion for the frictional interaction at the gravel-fabric interface, and it allows for incomplete anchorage. The results showed that the performance of high tensile modulus fabrics were affected to a higher degree by an incomplete anchorage than lower modulus fabrics. High modulus fabrics did not provide more reinforcing effects than low modulus fabrics in case of incomplete anchorage, as a consequence of the non-linearity of the soil-geotextile interaction mechanism.

Milligan *et al.* (1989) suggested a method for the design of unreinforced and reinforced unpaved roads on soft clay soils. The design was based on the concept that, when a vertical load is applied at the surface of the granular fill layer, it causes high vertical stresses, as well as horizontal stresses, under the loaded area. These stresses are held in equilibrium by shear stresses at the surface of the subgrade, and in unreinforced roads these shear stresses have a detrimental effect on the bearing capacity. The method presented moved away from a previous emphasis on membrane action of reinforcement,

and concentrated instead on the role of shear stresses on the clay-soil interface. The analysis demonstrated the function of reinforcement even at small surface deformations. The anchorage of the edge of the reinforcement was not viewed as important, but it was pointed out that high reinforcement stiffness may be necessary to realise the potential benefits.

Poorooshasb (1989) outlined a procedure for the analysis of geosynthetic-reinforced granular fill - soft soil system by using a transform function. The analysis indicated that the contribution from the geosynthetic in supporting the vertical load imposed by the footing was through the tensile stresses developed in the grid. These tensile stresses were developed through two different agencies. The first was due to a change of geometry of the grid due to the deformation of the system as a whole. The second was due to the dilational properties of the fill. The higher this factor was, the more was stretch of the grid and the more were tensile forces. As the overconsolidation ratio (OCR) for the granular fill (through compaction) increased, the efficiency of the reinforced soil system (defined as the ratio of the load for a predetermined settlement of the geosynthetic reinforced soil to the corresponding value for a soil system in its natural state) increased. It was found that at lower settlement level (less than 2.5 cm) the presence of geogrids had no effect at all. Further, it was found that the efficiency of the system decreases with an increase in the width of the footing. All the results presented were based on the assumption that the error introduced by neglecting the shearing traction on the upper surface of the fill was very small.

Vokas and Stoll (1989) used a continuum model to describe the response of a horizontally layered elastic system containing one or more reinforcing sheets that may be

located at any prescribed depth below the surface. The analysis was based on well-known equations for layered systems from the linear theory of elasticity. The effect of reinforcing was included by specifying the inter-layer boundary conditions on the basis of an analysis that was similar to that used in the classical theory of thin plates. The results presented for the case of axi-symmetric load represent a limiting case that should be approached by more general models when non-linear and inelastic effects are made small. Moreover, in many cases in which normal working loads are expected, the analysis will provide a good approximation for the trends that result from various changes in thickness and stiffness of the components.

Houlsby and Jewell (1990) presented a method of analysis of unpaved roads in which the tensioned membrane effect, which is in any case insignificant at small rut depths, was not considered. The method was based on the important concept that the principal function of the reinforcement was to carry outward shear stresses which would otherwise be applied to the soft clay subgrade. These shear stresses automatically put the reinforcement into tension, and the roughness of reinforcement became the important issue rather than anchorage. Design charts were presented which allow the necessary depth of granular fill, and the required reinforcement tension to be determined. The charts demonstrated that for certain combinations of parameters, a significant saving of fill thickness (or alternatively enhancement of bearing capacity) can be achieved by the use of reinforcement. It was suggested that if the tensile force in the reinforcement is to be developed at a small rut depth then a high reinforcement stiffness may be required. It should be noted that the suggested method of analysis is entirely in terms of monotonic vertical loading of a footing. The actual loading on a real road includes horizontal forces, due to vehicle cornering,

acceleration and deceleration. No account was taken of these forces in the design charts presented.

Sellmeijer (1990) presented a model for the behaviour of a soil-geotextile-aggregate system by combining the membrane action and lateral restraint (slab effect of the aggregate). In this model, the aggregate behaviour was modelled by elasto-plastic shear theory, the geotextile by membrane action and lateral restraint, and the subsoil by its bearing capacity. It was pointed out that the model is applicable to narrow low volume roads to wide parking pools. It is to be noted that this concept of modelling shows much smaller deflections than one where membrane action alone is considered and is suitable for design of paved roads.

Madhav and Ghosh (1990) extended the model suggested by Madhav and Poorooshasb (1988) to incorporate the non-linearity of soft soil. It was pointed out that a single layer of geosynthetic at the interface of the granular fill and the soft soil improves the load response, the improvement being more in case of very soft soil.

Ghosh (1991) further extended the works carried out by Madhav and Ghosh (1990), and Madhav and Poorooshasb (1989) to incorporate the non-linearity of shear stress-shear strain response of granular fill along with the application for multiple layers of geosynthetic reinforcement. While describing the characteristics of the suggested model, Ghosh pointed out that under large load, assumption of only horizontal shear stress transfer at the fill-geosynthetic interface for the inclined orientation of the geosynthetic reinforcement in the model may give erroneous results. This will surely happen because under large load, vertical shear stress transfer may be significant at the fill-geosynthetic interface.

Poorooshasb (1991a) studied the effect of a cavity which may appear in the subgrade, at some stage after construction of a reinforced fill. This study treated the problem as an equilibrium problem which is a more realistic representation of the actual case as majority of the works treated the situation as an instability problem. It was pointed out that the analysis is not exact, utilising only a kinematically admissible displacement field. However, since all the static boundary conditions except one were satisfied, the results obtained may be very close to the exact solution. The boundary condition that was not satisfied was at the surface of the fill where the analysis predicts a measure of shear stress. The magnitudes of these unbalanced stresses are very small and will not introduce errors of any significance.

Poorooshasb (1991b) demonstrated the usefulness of a transform function in obtaining solution to the problems of geosynthetic reinforced granular mats on weak subgrades. The class of problems that can be handled by the presented analysis include; (i) mats supporting point loads, uniformly or symmetrically distributed loads, (ii) mats bridging over voids appearing in the subgrade after construction and (iii) mats placed over nonuniform forms of ground subsidence. The solution presented was a kinematic solution which satisfied most, but not all, of the static boundary values of the problem. For example, in the vicinity of the reinforcements a discontinuity in the stress field was introduced which if it were of a finite value it would not be permissible. The kinetic field employed a fundamental hypothesis which stated that all originally vertical material planes remained vertical and did not undergo a change of dimension. The hypothesis was purely based on experimental evidence.

Pitchumani (1992) presented an analytical model using the elastic continuum approach to study the interactions between the reinforcements and the soil, and to predict the reduction in surface settlements due to reinforcements below a loaded area, at depth. The mechanisms considered were the shear and the normal stress interactions. The analysis was carried out for two types of reinforcements: rigid strips and flexible reinforcement. It was pointed out that the normal stresses resulting from vertical stress interactions for the strips as well as the sheets are much higher than the shear stresses resulting from the horizontal stress interactions. The optimum length of strips is 2 to 2.5 times the width of the loaded area.

Dixit and Mandal (1993) applied a variational method to determine the bearing capacity of geosynthetic-reinforced soil. In this method, the shape of the failure surface and the distribution of the normal stress over it were determined by the use of minimising theorems of variational calculus. Results of the variational method compared well with the experimental results of other investigators and showed a promising trend. The analysis carried out for determining the bearing capacity of shallow strip foundations loaded vertically and placed on geosynthetic-reinforced sand showed that the shape of the critical rupture surface is a log spiral and the shape of the rupture surface depends on both cohesion and the angle of internal friction. However, the approach is valid only for shallow reinforcement..

Espinoza (1994) presented a general expression for evaluating the increase of bearing capacity due to membrane action based on strict equilibrium conditions. Particular expressions of the average membrane action were obtained assuming constant and variable strain deformation of the membrane. Circular and parabolic shapes were used to simulate

the geotextile deformation. It was shown that independent of the model used and geotextile deformation shape assumed, comparable values for membrane support for relatively small rutting ratios were obtained. On the contrary, for large rutting ratios, the choice of the membrane support model and geotextile deformation shape had significant influence on the results, the latter being the most influential one. Models assuming the geotextile deformation to be circular furnished the larger values of the membrane support as compared to the parabolic ones. It was suggested that under large deformations, models using parabolic fabric deformation, such as the Giroud and Noiray's model, should be used.

Ghosh and Madhav (1994) extended the work of Madhav and Ghosh (1990) by incorporating the non-linearity of shear stress-shear strain response of granular fill and considering horizontal shear stress transfer at the fill-geosynthetic interface. The results indicated the improvements in the settlement behaviour of the geosynthetic reinforced granular fill-soft soil system subjected to uniformly distributed strip loading. The improvements in the settlement behaviour were significant with respect to stiffness of granular fill, when the soil is softer, and with respect to interfacial friction, when the fill material is less stiff. The improvement in settlement response due to reinforcement was of the same order, and over and above the effect of granular fill.

2.4 NUMERICAL WORKS

Brown and Poulos (1981) demonstrated how a finite element model of reinforced earth can be used to investigate the increase in bearing capacity and stiffness of a foundation due

to the placement of reinforcement in the soil. The finite element analysis was briefly summarised, and then used to examine the effect of reinforcement on the load-settlement behaviour of a strip foundation. It was shown that the improvement in foundation performance depends on both the number of reinforcing layers and on the concentration (surface area per unit width of footing) of the reinforcement. A footing on a reinforced soil mass overlying a cavity or a very soft zone was also analysed, and the reinforced soil was shown to result in a significant improvement in footing performance. It was concluded that the quantity of reinforcement necessary to produce a significant increase in bearing capacity is high. Since the limit of the reinforcement-soil bond was reached at an early stage, it appeared that the area of the reinforcement-soil interface (rather the stiffness of the reinforcement) is a significant factor.

Andrawes *et al.* (1982) described the finite element method of analysis involving discrete representation of the different constituents within soil-geotextile systems. The method was applied to the prediction of the behaviour of a footing resting on dense sand with or without a single layer of geotextile placed at different depths in various tests. It was shown that if the soil properties can be correctly represented in the soil elements, good correlations between predicted and measured data is obtained up to about 85% of peak load. Beyond this, the finite element method is inappropriate as local failures in the soil occur which can not be accommodated in the finite element procedures. It was concluded from the measured and predicted data that the influence of geotextile on the load-settlement behaviour of the strip footing is very limited up to settlements equal to approximately 8% of the footing breadth. This suggests that up to that level of settlement, strains in the soil are insufficient to mobilise significant tensile load in the geotextile.

Love *et al.* (1987) developed a finite element programme to handle large displacements and strains induced in the physical models of geosynthetic-reinforced granular fill - soft soil system with geosynthetic reinforcement at the fill-soft soil interface. The subgrade was modelled as an elastic-perfectly plastic material with limiting shear stress equal to undrained cohesion, c_u . The fill material was modelled as an elastic-frictional material obeying the Matsuoka yield criterion (Matsuoka, 1976). Reinforcement was modelled by three noded line elements of appropriate stiffness that conformed with the six noded triangular soil elements to either side. The reinforcement was treated as perfectly rough, so that failure occurred in soil elements adjacent to the reinforcement rather than the interface. It was suggested that with some confidence the formulation can be used to perform accurate predictions of full-scale structures. However, before applying the suggested finite element formulation to full-scale structures, its validity must be checked for large-scale model tests in field.

Rowe and Soderman (1987) reviewed the application of finite element techniques for the analysis of reinforced embankment behaviour. Details such as the choice of finite element and constitutive models as well as validation of finite element results against benchmark solutions were discussed. The use of plasticity solutions developed for a rigid footing, for estimating the maximum effect of reinforcement was also illustrated. It was demonstrated that the failure height for a reinforced embankment is related to the modulus of the reinforcement. As might be expected, the maximum force mobilised in the geotextile at failure increased with increasing geotextile modulus. However, because the deformation pattern in the soil also changed with increasing modulus (due to different levels of plasticity in the soil), these forces did not correspond to a unique strain that could

be generalised as a standard limit on strain to be used in limit equilibrium analyses. It was demonstrated from both field evidence and theoretical analysis that the reinforcement played a relatively small role at low load levels since the soil was essentially elastic. Significant strain in the geotextile began to develop with increasing plasticity and in fact most of the strain was developed after a contiguous plastic region is developed in the soil, since beyond this point the reinforcement was all that prevented collapse from occurring. Hence, the strains developed in the reinforcement for a given embankment height will largely depend on the height of the embankment relative to the height at which contiguous plasticity occurs, and hence will be sensitive to the magnitude and distribution of the actual shear strength in a soil deposit.

Koga *et al.* (1988) described the finite element analysis of soil reinforcement system consisting of geogrids used for two cases: (i) an embankment on soft soil and (ii) a footing. Their study revealed that the use of geogrids as soil reinforcements reduced the tensile stresses in the weak subsoil for the case of embankments and did not influence the settlement characteristics of the embankment. However, the maximum settlement was considerably reduced both in the case of surface and embedded footings. It was concluded that geogrids are better than strips as soil reinforcements.

Poran *et al.* (1989) presented a design procedure for the evaluation of settlement of footing placed on geogrid-reinforced granular fill overlaying soft clay subgrade. The method was based on finite element analysis which included a visco-plastic model for soils and visco-elastic membrane elements to model behaviour of geogrid reinforcement. The results from the parametric study indicated the effect of geogrid reinforcement for improvement of the load-deformation behaviour of such system. The design procedure

presented is applicable to continuous and axi-symmetrical footing specially in cases where small settlements are allowed.

Burd and Brocklehurst (1990) carried out a finite element study to investigate the effect of variations in the reinforcement stiffness on the structural behaviour of a typical reinforced unpaved road deforming in plane strain under the action of a single monotonic load. The results showed that the variations in the reinforcement stiffness had a more significant effect on the magnitudes of the shear stresses acting on the upper and lower surfaces of the reinforcement. The increases in reinforcement stiffness caused substantial increases in the magnitudes of the interface stresses and the reinforcement force. The parametric studies carried out for unpaved roads for small rut depth applications showed that for static loading, there was little benefit to be gained from using excessively stiff reinforcement. At large rut depths, new mechanisms of reinforcement might begin to operate which gave rise to different considerations in the choice of reinforcement stiffness. However, this aspect was not discussed in detail.

Wu *et al.* (1992) carried out finite element analysis to investigate the effectiveness of using geosynthetic tensile reinforcements for strengthening two highway test embankments, 8.5 m and 14.6 m in height, constructed over weak and highly pervious foundations. Both embankments were reinforced with geogrid mats placed near their base and were constructed in two stages. The results indicated that the use of tensile reinforcements near the base of the embankments which were constructed on weak and very pervious foundations had little effect on reducing vertical settlements of the embankments. This was because the foundations had large void ratios and were highly compressible, which resulted essentially in a one-dimensional deformation condition with

small lateral strains throughout the embankment-foundation systems. It was pointed out that the use of geogrid reinforcement might have served to bridge over weak zones likely to be present in the underlying foundations, and thereby could have conceivably alleviated local differential settlements. It was pointed out that the effectiveness of tensile reinforcements for reducing settlements of an embankment constructed over a weak foundation depended primarily on the deformation characteristics of the embankment. If an embankment with no reinforcement is expected to experience significant lateral spreading in the underlying foundation, use of tensile reinforcement may be effective for reducing overall differential settlements and increasing embankment stability. On the other hand, if little lateral straining in an embankment is expected, use of any tensile reinforcements would have little effect in reducing the settlements.

Otani *et al.* (1994) carried out the bearing capacity analysis of geogrid reinforced foundation ground using rigid plastic finite element method. In order to take into account the reinforcing effect in the analysis, a composite type model including geogrid and the surrounding soil was proposed. It was concluded that the bearing capacity of geogrid foundation ground increased as the depth and length of the reinforcement were increased, but there existed an optimum depth in order to mobilise the maximum reinforcing effect.

2.5 EXPERIMENTAL WORKS

Binquet and Lee (1975b) carried out about 65 model strip footing bearing capacity tests on sand foundations reinforced with strips of aluminium foil. The results of these tests were relatively consistent in showing that the load settlement and the ultimate bearing

capacities of the footings could be improved by a factor of about 2-4 times above the same load settlement or bearing capacity of an unreinforced soil for otherwise identical conditions. The bearing capacity continued to improve with increasing number of layers up to at least 6 to 8, beyond which little additional improvement was suggested by the data. The more usual observed modes of failure for an effective reinforced earth slab involved either ties pullout or ties breaking. Tie pullout failure generally occurred with lightly reinforced slabs, N<2 or 3, whereas tie breaking, which occurred in the uppermost layers, was generally associated with heavily reinforced slabs, N>4. The observed variations in behaviour of footings on reinforced soil from the classical slip surface bearing capacity theory indicated that a different failure mechanism applied and a corresponding different theory was probably appropriate for analysing the bearing capacity of the reinforced earth slabs than the classical bearing capacity theories.

Akinmusuru and Akinbolade (1981) conducted laboratory-scale bearing capacity model tests on square footing on a homogeneous sand bed reinforced with strips of a local rope material. Results obtained showed that the bearing capacity of a footing depended on horizontal spacings between strips, vertical spacing between layers, depth below the footing of the first layer, and the number of layers of reinforcement. It was shown that depending on the strip arrangement, ultimate bearing capacity values could be improved by a factor of up to three times that of the unreinforced soil. However, practical considerations could limit suitable arrangements to bearing capacity improvement factors of about two. However, it was necessary to look into the long-term effect of this type of reinforcement material on the performance of actual rural housing or road design projects.

Kinney (1982) performed small scale laboratory tests to study the effect of a geotextile on a soil-geotextile-aggrgate system used as a unsurfaced roads. The test demonstrated that a geotextile disc slightly larger than the loading plate had a very significant reinforcing effect on the reinforced soil system; whereas, a disc identical in size to the loading plate did not appear to have any effect. The effect of a limited expanse of geotextile diminished as the thickness of the aggregate increased. It was also pointed out that significant deformation appeared to be required for the geotextile to act as a reinforcement, and additional deformation accentuated the beneficial effects.

Sowers *et al.* (1982) investigated the mechanism of failure of aggregate surfaced roads on very weak subgrades by (i) study of selected road failures, (ii) small scale load tests, (iii) large scale static load tests, and (iv) full scale, moving vehicle loading. It was observed that the geotextile provided tensile restraint for the aggregate, which enhanced the load spreading to the subgrade. This reduced elastic deflection with a light load and increased the load causing failure after one or two load repetitions. The geotextile separated the porous aggregate and the underlying soft subgrade, preventing soil intrusion that destroyed the aggregate load spreading upon repeated loading. After repeated heavy loading plus rut repair, the geotextile sagged below the wheel path and bulged adjacent to it. Load support was more than doubled by catenary support below the wheel, catenary restraint of the subgrade bulge, and by enhancing the aggregate load spreading.

Fragaszy and Lawton (1984) conducted a series of laboratory model tests designed to determine the influence of soil density for a wide range of relative densities (D_r = 51% - 90%) and reinforcing strip length on the load-settlement behaviour of reinforced sand. The results indicated that when bearing capacity ratio was calculated at a settlement equal to

10% of the footing width, the bearing capacity ratio was independent of soil density. When calculated at a settlement of 4% of the footing width, the percent increase in bearing capacity appeared to be less for loose sands than for dense sands. Failure of rectangular footings on dense reinforced sand occurred at a larger settlement than an identical footing on unreinforced sand at the same density. As strip length increased from 3 to 7 times the footing width, the bearing capacity ratio increased rapidly. It was pointed out that equations developed for calculation of bearing capacity of reinforced earth slabs by Binquet and Lee (1975b) were very sensitive to the magnitude of the soil-reinforcing strip friction coefficient.

Guido *et al.* (1985) carried out laboratory model tests to investigate the bearing capacity of a square footing on a geotextile reinforced sand of medium compact density (relative density = 50%). For the tests performed, the bearing capacity of the sand reinforced with geotextiles was increased by a factor greater than three. Their tests used a high-modulus nonwoven melt-bonded geotextile and varied a number of parameters. The results indicated that the multiple layers (up to three) produced beneficial results, but only after a measurable settlement had occurred. This was to be expected, since the fabric-soil system must deform before its reinforcing benefit was realised. It was reported that the increase in the ratio of width of footing to the width of reinforced zone beyond 3, there was little change in the bearing capacity ratio.

Dembicki *et al.* (1986) conducted model tests of rigid strip foundation on subsoil reinforced by horizontally placed geotextile. The two-layer subsoil was used, i.e., mud covered by sand. From the results presented, the influence of 5 parameters can be extracted and analysed separately for the similar soil conditions. They were: thickness of

sand layer, length of geotextile band, type of geotextile, inclination of load, eccentricity of load. It was pointed out that the effect of reinforcement was observed at big deformations only, with maximum influence on settlement, s, at $B/2<s<B$ (B = foundation width).

Guido *et al.* (1986) conducted a series of laboratory plate loading tests using either geogrids or geotextiles as reinforcement in sand. The results showed that the bearing capacity of an unreinforced earth slab could increase substantially with the insertion of either type of reinforcement. For geotextiles to function properly as reinforcement, friction must develop between the soil and the reinforcement to prevent sliding, whereas for grids, it was the interlocking of the soil through the apertures of the grid that achieved an efficient anchoring effect. Therefore, it was suggested that fixed shear box tests and pull-out tests are required to be run to determine the failure mechanisms of geotextiles and geogrids, respectively. It was pointed out that geogrids were specifically used as a reinforcement material, whereas geotextiles had other functions, such as separation, drainage, and filtration. It was concluded that geogrids were superior form of reinforcement owing to the interlocking of the soil with the grid membrane. However, it was pointed out that the type of reinforcement used may be very site specific because in some situations, the reinforcement may also have to function as a separator.

Love *et al.* (1987) demonstrated the effectiveness of geogrid reinforcement, placed at the base of a layer of granular fill on the surface of soft clay by small scale model tests . In the tests, monotonic loading was applied by a rigid footing, under plane strain conditions, to the surface of reinforced and unreinforced systems, using a range of fill thickness and subgrade strengths. The deformations of the subgrade and of the geogrid reinforcement were measured from photographs. From these measurements the different mechanisms of

failure in the unreinforced and reinforced system were established. Performance of reinforced systems was found to be superior even at small deformations, owing to the significant change in the pattern of shear forces acting on the surface of clay, brought about by the presence of the reinforcement. It was found that the membrane action of the reinforcement becomes significant at large deformations only.

Sakti and Das (1987) investigated the ultimate bearing capacity of a model strip foundation resting on a saturated soft clay internally reinforced with geotextile layers in the laboratory. It was found that geotextile layers placed under a foundation within a depth equal to the width of the foundation had some influence on the increase of the short-term ultimate bearing capacity. It was suggested that for maximum efficiency, the first layer of geotextile should be placed at a depth of about 0.4 times the width of the foundation. It was pointed out that the minimum length of the reinforcing geotextile layers for maximum efficiency was about four times the width of the foundation.

Valsangkar and Holm (1987) reported the results of experimental research dealing with interaction of lightweight aggregate (unit weight = 13 to 15 kN/m^3) and geotextiles overlying peat subgrades. Variables investigated were: differing aggregate types and densities, thickness of the aggregate layer, and geotextile types. The results indicated that the overall roadbed stiffness was unaffected when lightweight aggregate was used instead of normal weight aggregate, for small deflections and initial load application. The results also showed that the reinforcing role of a geotextile was insignificant during the initial stages of load application.

Kim and Cho (1988) investigated the effects of geotextile reinforcement on bearing capacity and deformation of soil foundation in view of the distance of footing from

geotextile layer and the footing embedment ratio. Tests were carried out under partial drainage condition. From experiments, it was found that the contribution of geotextile to the bearing capacity is high as the distance of the footing from geotextile layer reduced, as the embedment depth of footing increased, and as the settlement of footing increased. It was pointed out that the ratio of sand layer depth on soft clay layer to strip footing width, H/B, which gave the greatest beneficial effects of geotextile, fell between 0.5 and 1.0 for the settlements where s/B was less than 1.0. It was observed that the failure of the reinforced foundation occurred at high bearing pressure in a large deformation mode of circle due to geotextile effect while the unreinforced foundation soil failed in a small deformation mode at low bearing pressure.

Miura *et al.* (1988) carried out model tests and finite element analysis of reinforced pavements by polymer grid to investigate the effect of polymer grid in suppressing a non-uniform settlement of asphalt pavement constructed on the soft clay ground. The model and field test results showed that polymer grid placed at the interface of subbase and subgrade effectively suppressed the settlement under repeated loading. Finite element analysis, in which joint element was introduced, indicated that the polymer grid was not effective in suppressing the surface settlement. This contradiction suggested that an important function of polymer grid in the base might be the interlocking effect which was not considered in the numerical modelling. According to the field data obtained, one and three months after the construction, the modulus of subgrade reaction, k, of the reinforced pavement was smaller than that of conventional pavement. This might come partly from overestimation of polymer grid function and partly from insufficient compaction of the base owing to the action of the polymer grid. It was suggested that to make polymer grid

useful as a reinforcing material of the base in a pavement on the soft ground, compaction works should be performed carefully, not leaving gap between polymer grids and underneath layers.

Verma and Char (1988) carried out model tests to show the beneficial effects of using vertical reinforcing rods for sand subgrades. It was pointed out that the improvement was a function of the spacing, diameter, roughness and extent of the reinforcing element. The advantage of this method is that relaying of the subgrade is not required as in the case of horizontal reinforcements. However, this method cant not be used for geosynthetic reinforced soils.

Das (1989) presented laboratory model test results for ultimate bearing capacity of strip and square shallow foundations supported by a compact sand layer underlain by a soft clay with and without a geotextile at the sand-clay interface. The test results indicated that with the use of geotextile, the critical value of the H/B ratio at which the maximum bearing capacity ratio occurred was about 0.75 for strip foundations and about 0.5 for square foundations, where B is the foundation width , and H is thickness of compacted sand layer below the base of the footing. These values of H/B were about half of those obtained when a geotextile was not used at the interface.

Gorle and Thijs (1989) carried out laboratory model tests with prestressed geosynthetics and with granular materials either confined by geotextiles or reinforced with short fibres. In tests, the anchorage of the prestressed geosynthetic was released which resulted in a horizontal confining stress of maximum 20 kPa building up in the granular material. It was observed that for low deformation soil-sand systems (CBR of soil between 3 and 6%), the prestressing of geosynthetics limits the total settlement of the system and

increased the cyclic bearing capacity, especially when the layer thickness was less than or equal to the radius of the loading surface. The tests on granular materials reinforced with short synthetic fibres showed a considerable influence on the cohesion, the angle of internal friction and the modulus, depending not only the amount of the fibres but also on their nature and their dimensions in relation to the pore size distribution of the granular material.

Selvadurai and Gnanendran (1989) presented results of a series of experimental investigations conducted to determine the manner in which the performance of a footing located at the crest of a sloped fill could be influenced by the presence of a reinforcing layer within the body of the fill. The results indicated that the load-carrying capacity of a footing on a sloped fill could be improved in excess of 50% by incorporating geogrid reinforcement. When considering the ultimate load-carrying capacity, the optimum location for the geogrid reinforcement occurred at a depth between 0.5 and 0.9 times the width of the foundation. The primary properties of a geogrid that governed its effectiveness in improving the load-carrying capacity of the sloped fill were identified as the aperture size, the modulus of elasticity, and the tensile strength. It was found that the location of the geogrid layer at a depth greater than twice the width of the footing did not lead to any improvement in either the load-carrying capacity or the stiffness of the footing on a sloped fill.

Koerner (1990) reported the work carried out at Drexel University's Geosynthetic Research Institute on soft, compressible, fine-grained soils at saturations above their plastic limit. The load-settlement curves were presented for circular footing on soft saturated clay

silt reinforced with geotextile reinforcement. Some improvement in bearing pressure was noted throughout, but only at large deformations is the improvement noteworthy.

McGown *et al.* (1990) conducted an extensive programme of monotonic loading footing tests by incorporating bamboo rods, which possessed both bending and tensile stiffnesses, together with a layer of geotextile at the sand-clay interface. Results showed that large increases in bearing capacity might be achieved even at low deformations. This was quite different from the case where only a geotextile was used. Observations revealed that the deformation mechanisms of the system were quite different. When bamboo rod reinforcements, having tensile and bending stiffnesses were used in conjunction with the geotextile, the bamboo rods provided both bending and tensile reinforcement to the sand, whilst the geotextile acted as a separator and filter between the sand and clay. However, it also acted as a tension membrane between the bamboo poles, and as a cushion beneath the poles thereby reducing the localised stresses in the clay. The mathematical model suggested to allow a design method was successfully tested against laboratory data but it must be checked against full-scale field trials on unpaved, unbound roads, before it can be used with confidence.

Mandal and Sah (1993) carried out bearing capacity tests on model footings on clay subgrades reinforced with geogrids placed horizontally. Tests results showed that the effectiveness of geogrid reinforcement increased the bearing capacity of clay subgrades, with improvements being observed at nearly all levels of deformation. Results indicated that the maximum percentage reduction in settlement with the use of geogrid reinforcement below the compacted and saturated clay was about 45% and it occurred at a distance of 0.25 B (B being the footing width) from the base of the square foundation.

Omar *et al.* (1993) carried out laboratory model test for the ultimate bearing capacity of strip and square foundations supported by sand reinforced with geogrid layers. Based on the model test results, the critical depth of reinforcement and the dimensions of the geogrid layers for mobilising the maximum bearing-capacity ratio were determined and compared. It was concluded that for development of maximum bearing capacity, the effective depth of reinforcement was about 2B for strip foundations where, B was width of strip footing and 1.4B for square foundations where B was length of each side of the square.

Puri *et al.* (1993) conducted a number of laboratory model tests on a square surface foundation supported by sand with and without critical geogrid reinforcement. The foundation was subjected to an initial allowable static load. A cyclic load was then superimposed on the static load, and the permanent settlement of the foundation during cyclic loading was monitored. Based on the model test results, it was concluded that the ultimate permanent settlement increased with the increase of the amplitude of the cyclic loading. It was also reported that the ultimate permanent settlement was markedly reduced due to geogrid reinforcement.

Abduljauwad *et al.* (1994) conducted a research program to assess the performance of *sebkha* subgrade (a saline soil frequently occurring along the western shore of the Arabian Gulf, associated with many geotechnical problems that emerge principally because of their high salt content and their susceptibility to strength loss and collapse upon saturation) in the laboratory using conventional and soil-fabric-aggregate (SFA) systems under static and dynamic loading conditions. Results presented indicated that the use of geotextiles significantly enhanced the inferior properties of *sebkha* subgrade, particularly when the SFA system was saturated. The contribution of geotextiles to the performance of SFA

systems diminished when the subbase thickness increased. It was reported that the results of the index laboratory tests on the small-scale model had been used by local municipalities for road constructed on *sebkha* subgrade in the eastern province of Saudi Arabia.

Das and Shin (1994) carried out laboratory model tests to determine the permanent settlement of a surface strip foundation supported by geogrid-reinforced saturated clay and subjected to a low-frequency cyclic load. In conducting the tests, the foundation was initially subjected to an allowable static load. The variation of the cyclic load was then super-imposed over the static load. Based on the test results, it was concluded that for a given amplitude of the cyclic load intensity, the maximum permanent settlement increased with the increase in the intensity of the static load and for a given intensity of static loading, the maximum permanent settlement increased with the increase in the amplitude of the cyclic load intensity. It was also pointed out that full depth geogrid reinforcement might reduce the permanent settlement of a foundation by about 20% to 30% to one without reinforcement.

Floss and Gold (1994) examined the improvement of the bearing and deformation behaviour by means of a geosynthetic reinforcement placed at the base of a layer of granular fill on the surface of soft clay (called reinforced two-layer system). It was tried to present the reasons for improvement by carrying out both site tests and finite element analysis. It was pointed out that with reinforcement the granular layer was able to transmit shear forces on an essentially higher level without collapsing. The function of (better) load spreading was maintained also on a high deformation level.

Khing *et al.* (1994) presented laboratory model test results for the ultimate bearing capacity of a surface strip foundation supported by a strong sand layer of limited thickness

underlain by a weak clay with a layer of geogrid at the sand-clay interface. Based on the model test results presented, it appeared that the optimum height of the strong sand layer should be about two-thirds that of the foundation width for obtaining the maximum benefit from the geogrid reinforcement in increasing the ultimate bearing capacity. Since the results have been presented by conducting the tests at one relative density of compaction of sand and one undrained shear strength, it will be more logical to verify the results for several other sets of relative density of sand and undrained shear strength.

Manjunath and Dewaikar (1994) carried out model footing tests to determine the effect of a single layer of geosynthetic reinforcement on the bearing capacity of shallow foundations. The tests were conducted with square footings resting on compact sand layer overlying a soft clay subgrade. It was concluded that the size of the footing did not have any significant effect on the performance of the footings on reinforced soil beds. It was also pointed out that the primary properties of the reinforcement material that affect the performance of footings on reinforced soil beds were their tensile strength, elastic modulus and aperture size. It was suggested that if one had to place a footing on sand above soft clay subgrade, the geotextile would be most ideally suited than geogrids.

Nishida and Nishigata (1994) carried out laboratory tests by applying cyclic load on the surface of the pavement model in cylindrical mould to evaluate the separation function of the geotextile and to find out its relationship with reinforcement function. It was pointed out that the reinforcement was a prime function when the ratio of the applied stress on the subgrade soil to the shear strength of the subgrade soil (σ/c_u) was high, however, the separation could be an important function when the ratio was low.

Yetimoglu *et al.* (1994) investigated the bearing capacity of rectangular footings on geogrid-reinforced sand by performing laboratory model tests as well as finite element analyses. Both the experimental and analytical studies indicated that there was an optimum reinforcement embedment depth at which the bearing capacity was the highest when single-layer reinforcement was used. Also, there appeared to be an optimum reinforcement spacing for multi-layer reinforced sand. The analysis, for the conditions investigated, indicated that increasing reinforcement stiffness beyond 1000 kN/m would not bring about further increase in the bearing capacity.

2.6 CONCLUSIONS

In the previous sections, a critical review of available literature pertaining to the near-surface reinforced foundation soil are presented. From the results of a large number of model tests conducted till very recently and also from the results presented through several analytical and numerical studies of geosynthetic-reinforced granular fill - soft soil system, it is observed that the geosynthetics, particularly geotextiles and geogrids, show their beneficial effects as reinforcements only after relatively large settlements (Andrawes *et al.*, 1982; Milligan and Love, 1984; Guido *et al.*, 1985; Dembicki *et al.*, 1986; Rowe and Soderman, 1987; Valsangkar and Holm, 1987; Madhav and Poorooshasb, 1988; Poorooshasb, 1989) which may not be a desirable feature for shallow footings, paved and unpaved roads, embankments etc. Hence, the need has been felt for a technique which can make the geosynthetics more beneficial without the occurrence of large settlements.

Prestressing the geosynthetic reinforcement can be one such technique which can reduce the settlements of the geosynthetic-reinforced foundation soil significantly. The idea

of prestressing has been recognised in the past (Barvashov *et al.*, 1977; Aboshi, 1984; Watary, 1984; Broms, 1987; Gorle and Thijs, 1989; Hausmann, 1990; Koerner, 1990). However, hardly any experimental study has been carried out on the prestressing of geosynthetics and its effect on settlement characteristics of geosynthetic-reinforced granular fill - soft soil system. In fact,, most of the earlier workers have realised the importance of prestressing the geosynthetics but analytical work is scarce.

It is noted that most of the existing simple foundation models, especially mechanical models, consider only the horizontal shear stress transfer mechanism at the soil-geosynthetic interface and hence their applications are restricted to problems involving infinitesimal deformations only. For dealing with problems involving relatively large deformations, both the horizontal and the vertical shear stress transfer mechanisms should be considered.

It is also noticed from the literature review, that the compressibility of the granular fill has been neglected in most of the studies carried out so far which may have significant effect on the settlement behaviour, especially in those situations where a layer of loose compressible granular fill is used. It is also observed that a simple model for estimating the settlements of the geosynthetic-reinforced granular fill - soft soil system at different stages of consolidation of the soft soil has not yet been developed. In fact, there is a need of development of a foundation model to incorporate these parameters in a simple way so that one can easily estimate the settlements by considering most of the factors governing the behaviour under specific field situations.

CHAPTER 3

DEVELOPMENT OF A FOUNDATION MODEL

3.1 INTRODUCTION

One of the common approaches to solve many geotechnical engineering problems involving interaction between the structural foundation and supporting soil subgrade is to idealise the soil subgrade by mechanical foundation models, such as Winkler model, Filonenko-Borodich model, Pasternak model etc. (Kerr (1964), Selvadurai (1979) and Horvath (1989)). In their original development, these models idealised the behaviour of one soil layer only. However, the concepts involved in these models are fairly general in nature and can be applied to idealise the behaviour of two soil layers, such as a granular fill on soft foundation soil. For example, in Pasternak foundation model, which consists of an incompressible shear layer and Winkler springs, the shear layer can be assumed to represent the shear characteristics of the granular fill by neglecting its compressibility and the Winkler springs can be assumed to represent the compressibility of the soft soil by neglecting its shear characteristics.

The models for general soil behaviour as described above can not be used as such for idealising the behaviour of geosynthetic-reinforced granular fill - soft soil system (Fig. 3.1) because the geosynthetic reinforcement introduced into the soil works through

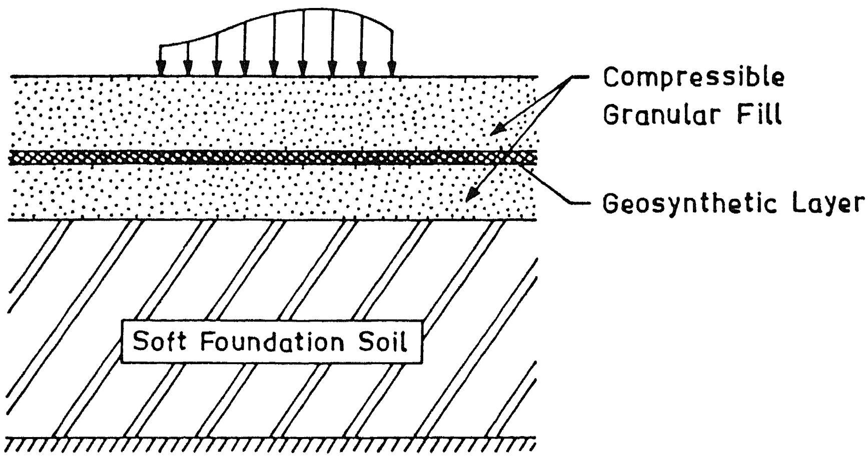


Fig. 3.1. Geosynthetic-reinforced granular fill - soft soil system.

several mechanisms such as, membrane effect, confinement effect, separation effect and anchoring effect. Hence, in recent years, mechanical foundation models for geosynthetic-reinforced soil (Madhav and Poorooshasb, 1988, 1989; Madhav and Ghosh, 1990; Ghosh, 1991; Ghosh and Madhav, 1994) have been developed and these models have been described in the earlier chapter.

In the present chapter, a mechanical foundation model for geosynthetic-reinforced granular fill - soft soil system is developed to incorporate several factors which govern the settlement response of the system. During development of the model, it is tried to present the basic concepts of the parameters to be incorporated.

3.2 SHEAR STRESS TRANSFER MECHANISM

In the existing mechanical foundation models for geosynthetic-reinforced granular fill - soft soil system (Madhav and Poorooshasb, 1988, 1989; Madhav and Ghosh, 1990; Ghosh, 1991; Ghosh and Madhav, 1994), the geosynthetic reinforcement is represented by a foundation model element using a rough elastic membrane. The granular fill and the soft subgrade are idealised by Pasternak shear layer and a layer of Winkler springs respectively (Fig. 3.2). These models consider only the horizontal shear stress transfer at the

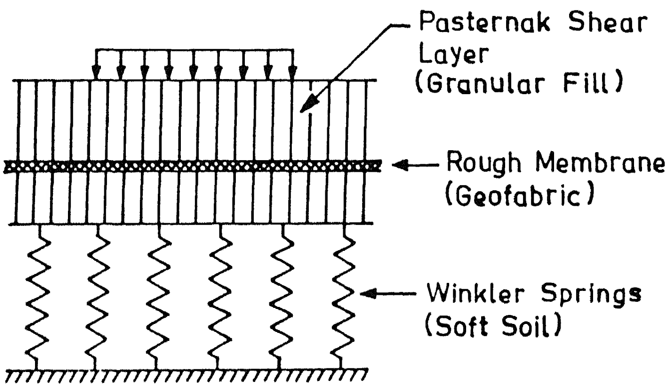
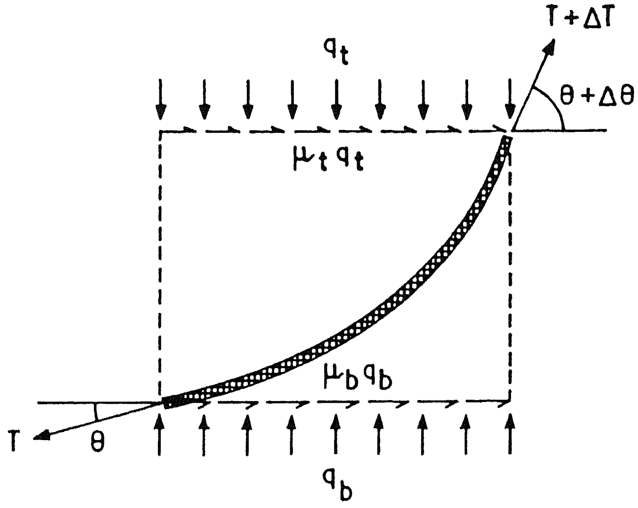


Fig. 3.2. Foundation model for granular fill - geofabric - soft soil system. (From Madhav and Poorooshasb, 1988.)

fill-geosynthetic interface and ignore the vertical shear stress transfer, even for inclined orientation of the membrane (Fig. 3.3). This consideration may limit their applications for settlement predictions of geosynthetic-reinforced granular fill - soft soil systems under the loaded structures in most of the situations existing in fields where large deformation occurs. Through several model tests and also from theoretical studies reported in literature (Chapter 2), it is quite evident that the geosynthetics, mainly geotextiles and

geogrids, show their beneficial effects only after relatively large settlements. This



q_t = vertical force interaction between the membrane and the upper shear layer
q_b = vertical force interaction between the membrane and the lower shear layer
μ_t = interfacial friction coefficient at the top face of the membrane
μ_b = interfacial friction coefficient at the bottom face of the membrane
T = tensile force per unit length mobilised in the membrane
θ = slope of the membrane
ΔT, $\Delta\theta$ = small increments in T and θ respectively

Fig. 3.3. Shear stress transfer mechanism adopted by existing mechanical foundation models for geosynthetic-reinforced granular fill - soft soil system.

fact suggests a need of further improvement of the model which would consider vertical as well as horizontal shear stresses transfer mechanism at the fill-geosynthetic interface.

In order to incorporate the vertical shear stress transfer mechanism at the fill-geosynthetic interface, it is assumed that the horizontal stress in the granular fill is also present. The horizontal stress may be induced as a consequence of applied vertical load on the surface of the granular fill. If the granular fill is placed under confined conditions in

the field and compacted by rollers, vibrating plates, or rammers, the horizontal earth pressure within the compacted fill is increased. When the compaction equipment moves away, the vertical pressure decreases to its normal overburden. The horizontal earth pressure also decreases somewhat as the compaction equipment moves away, but it remains above its precompaction value. Thus, the compaction represents a form of overconsolidation of the granular fill (Duncan and Seed, 1986; Poorooshasb, 1989). Duncan *et al.* (1991) reported that the horizontal earth pressure induced by compaction of granular materials like sand do not change appreciably with time unless confinement is changed.

In the light of the facts described above, a more general shear stress transfer mechanism is proposed considering both the horizontal and the vertical shear stress transfers at the fill-geosynthetic interface as shown in Fig. 3.4. The horizontal stress at the top and bottom

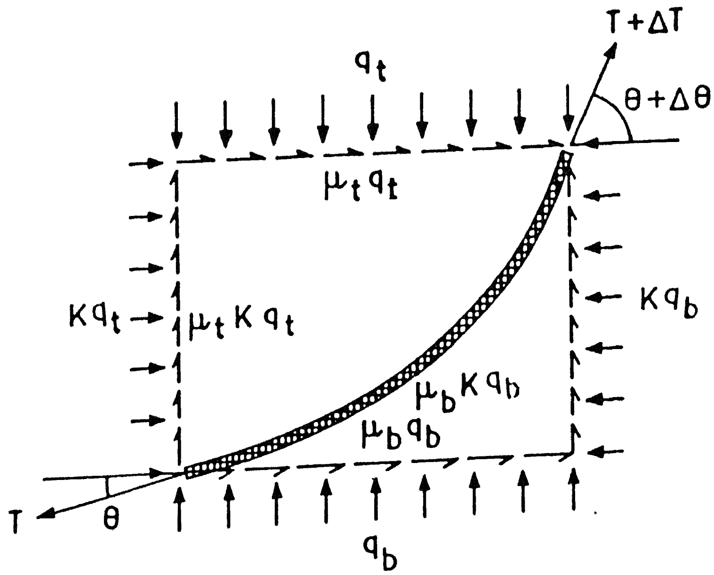


Fig. 3.4. The proposed shear stress transfer mechanism.

of the geosynthetic layer are Kq_t and Kq_b respectively. K is the ratio of horizontal to vertical stresses present in the granular fill. If the stresses developed within the granular fill are maximum in stress history, the reinforced granular fill is at a K_0 (coefficient of lateral stress at rest) state of stress as observed by Schlosser and Long (1972) in case of triaxial tests on reinforced sand. Hence in such situations, $K = K_0$ should be considered.

However, if granular fill is overconsolidated (through compaction), K will be greater than K_0 , and for any overconsolidation ratio (R_c) corresponding to particular field conditions, it may be estimated using the relationship suggested by Alpan (1967) as:

$$K = K_0 \, R_c^{\lambda} \tag{3.1}$$

where, λ is at rest-rest rebound exponent which was correlated with effective angle of shearing resistance (ϕ') for sands (Fig. 3.5). K_0 can be determined using the relation:

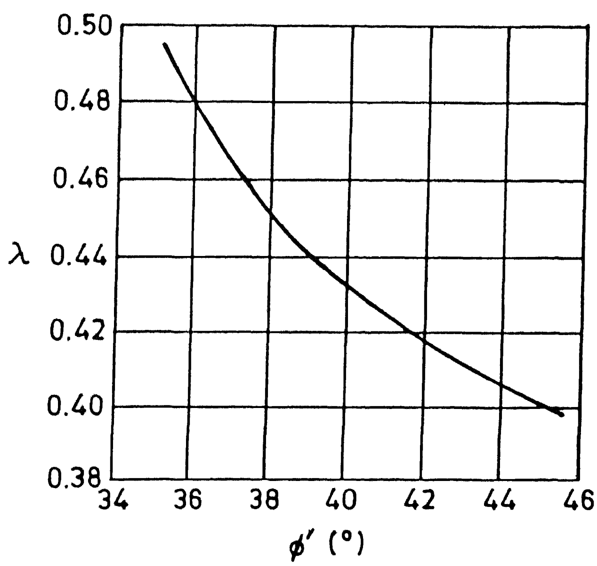


Fig. 3.5. The variation of the parameter λ with the angle of shearing resistance, ϕ', in sands. (From Alpan, 1967.)

$K_0 = 1 - \sin\phi'$, suggested by Jaky (1944). In eqn (3.1), the value of R_c to be substituted should be determined keeping in mind the following basic definition:

$$R_c = \frac{\text{Maximum stress induced in the granular fill through compaction}}{\text{Existing stress under the working load}} \quad (3.2)$$

There can be other possible means to obtain the value of K. Several works on compaction induced stresses are available in literature (Rowe, 1954 ; Alpan, 1967; Broms, 1971; Duncan and Seed, 1986; Duncan et al 1991). Similar to the relation (3.1), a relation was also suggested by Al-Hussaini and Townsend (1975) which my also be used.

3.3 PRESTRESSING OF GEOSYNTHETIC REINFORCEMENT

In the present work, the effect of prestressing the geosynthetic reinforcement on the settlement characteristics of reinforced soil is studied in a quantitative way by idealising the prestressed geosynthetic-reinforced granular - soft soil system by representing each sub-system by mechanical foundation model elements. The granular fill and the soft foundation soil is represented by Pasternak shear layer and Winkler springs respectively. The membrane used in the existing mechanical foundation models for geosynthetic reinforced soil is a rough elastic one while, the Filonenko-Borodich model for general soil uses a smooth elastic membrane under a constant tension in all horizontal directions to achieve the continuity between the individual springs in the Winkler model. Neither of these two foundation model elements can individually be used for representing the

prestressed geosynthetic properly. Hence, in the present work, these two foundation model elements are combined into one as a *stretched rough elastic membrane* to idealise the behaviour of a prestressed geosynthetic.

The foundation model for the prestressed geosynthetic reinforced granular fill - soft soil system is obtained by combining the stretched rough elastic membrane with Winkler springs and Pasternak shear layer as shown in Fig. 3.6. One can derive the equations governing the response of the model along with the expression for mobilised tension using shear stress transfer mechanism as shown in Fig. 3.4.

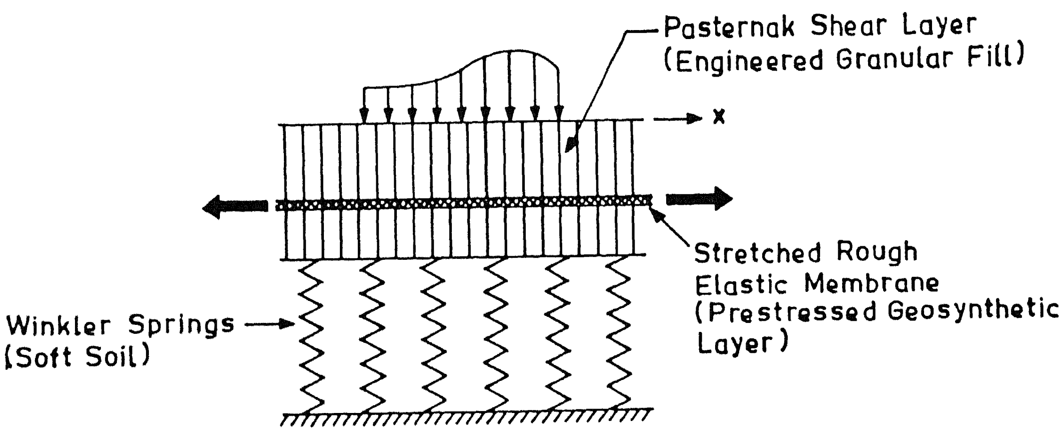


Fig. 3.6. Proposed foundation model for geosynthetic-reinforced granular fill - soft soil system incorporating prestress in the geosynthetic reinforcement.

3.4 COMPRESSIBILITY OF GRANULAR FILL

The compressibility of the granular fill may affect the settlement behaviour of the geosynthetic-reinforced granular fill - soft soil system. In most practical field situations, the granular fill may have some compressibility, although this is relatively small compared to the compressibility of the soft soil. It is appropriate to consider this aspect in situations

where the reinforced soil systems are used as a foundations for structures such as shallow footings, embankments etc. which need accurate settlement predictions from the point of view of their functional and structural requirements.

In most of the studies carried out so far, the compressibility of the granular fill in geosynthetic reinforced granular fill - soft soil system has been disregarded. No major effort is carried out in past to clearly bring the effect of compressibility of the granular fill on the settlement behaviour of the reinforced soil system. In mechanical modelling, the granular fill is generally idealised by the Pasternak shear layer which is an incompressible layer capable of deforming by vertical shear transfer only.

In the present work, to study the effect of compressibility, the model presented in Fig. 3.6 is modified as shown in Fig. 3.7 in which the compressibility of the granular fill is incorporated by attaching a layer of stiff Winkler springs to the bottom of Pasternak shear layer representing the shear characteristics of the granular fill. The spring layers

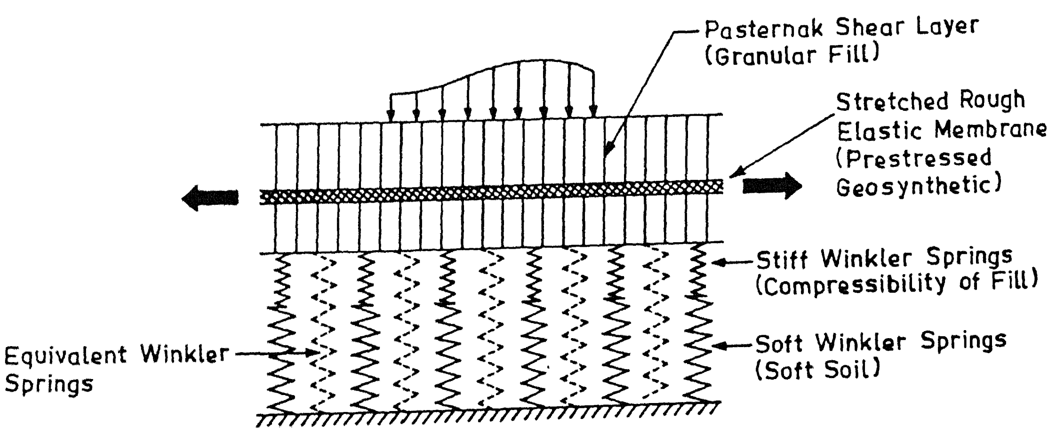


Fig. 3.7. Proposed foundation model for geosynthetic-reinforced granular fill - soft soil system incorporating the compressibility of the granular fill.

connected in series to represent the compressibility of the granular fill and the soft soil, are replaced by an equivalent spring layer shown by dashed lines. The modulus of subgrade reaction for the equivalent spring layer can be calculated as:

$$k_e = \alpha^* k_s \tag{3.3}$$

where, k_s is the modulus of subgrade reaction for the soft foundation soil, and α^* is given by:

$$\alpha^* = \frac{\alpha}{1+\alpha} \tag{3.4}$$

in which, α ($= k_f / k_s$) is a modular ratio, k_f being the modulus of subgrade reaction for the granular fill. Clearly, the modular ratio is a measure of relative compressibility of the granular fill and the soft foundation soil. One can select α considering the existing conditions in the field.

3.5 TIME-DEPENDENT BEHAVIOUR

If one wants to estimate the settlement of the geosynthetic-reinforced granular fill - soft soil system at any stage of consolidation of the soft foundation soil, it is not possible to get the result using the concepts presented above. For such a reinforced soft soil system, there is no mechanical foundation model which can be used directly to study the time-dependent mechanical behaviour. The representation of soft foundation soil by Winkler springs in mechanical foundation model makes the settlement response of the model to be

instantaneous. In fact, the soil, specially saturated clay under applied load does not undergo an instantaneous settlement, the consolidation process causes its settlement gradually for a long time.

At present, Terzaghi one dimensional consolidation theory is widely used in both the theoretical and the practical applications in geotechnical engineering. The consolidation behaviour of the soft clay, which is governed by Terzaghi consolidation theory, may be included in the model shown in Fig. 3.7. The improved model is shown in Fig. 3.8 in which, the soft foundation soil is represented by a spring-dashpot layer. The springs represent the soil skeleton and the dashpots represent the dissipation of excess pore water pressure in the voids of the foundation soil.

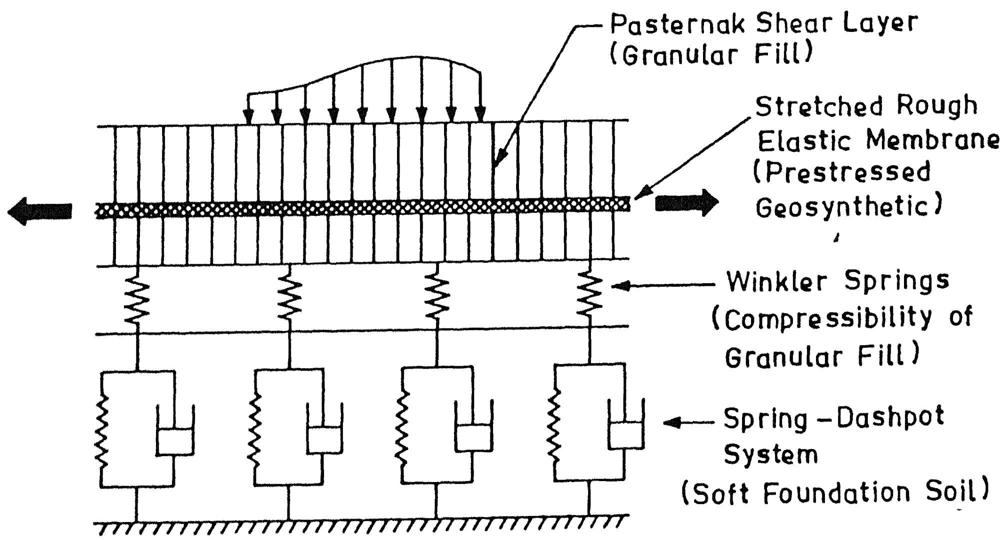


Fig. 3.8. Proposed foundation model incorporating several factors governing the settlement response of the geosynthetic-reinforced granular fill - soft soil system.

3.6 CONCLUSIONS

In this chapter, the basic concepts in the development of a mechanical foundation model for geosynthetic-reinforced granular fill - soft soil system are presented. The behaviour of fill-geosynthetic interface in shear is represented by a more appropriate shear stress transfer mechanism which considers both the horizontal and vertical shear stress transfers so that one can solve the problems involving large deformations. It is suggested that prestressing the geosynthetic reinforcement can be useful in making the geosynthetic reinforcement to be of much beneficial use before the occurrence of large settlements which are often observed in its applications. The proposed foundation model can be used to estimate the effects of prestressing the geosynthetic reinforcement on the settlement response. The effect of compressibility of the granular fill which has been ignored in most of the earlier studies, can be observed quantitatively using the proposed model. This model is also able to predict the settlement of the reinforced soil system under loaded structure at any stage of consolidation of the soft foundation soil. In fact, the proposed foundation model provides a simple means to study and to predict the settlement behaviour of geosynthetic-reinforced granular fill - soft soil system considering the effects of several key factors which have not been considered in earlier studies.

CHAPTER 4

SETTLEMENT ANALYSIS - PLANE STRAIN CASE

4.1 INTRODUCTION

In the present chapter, the equations governing the settlement response of the foundation model, developed in Chapter 3 to represent the geosynthetic-reinforced granular fill - soft soil system, are derived for a strip loading applied directly to the top of the granular fill in plane strain conditions. A detailed parametric study is carried out to bring out the effects of several parameters on the settlement characteristics of reinforced soil system. The variation of the mobilised tensile force in the geosynthetic reinforcement is also studied. The present results are compared with those computed using the existing mechanical foundation models, wherever it is possible

The numerical solutions are obtained by an iterative finite difference scheme and the results are presented in a nondimensional form. The presented results are discussed in the light of field applications of the geosynthetic-reinforced granular fill - soft soil systems as foundations.

equations governing its settlement response. In this model, the geosynthetic reinforcement and the granular fill have been represented by the stretched rough elastic membrane and the Pasternak shear layer respectively. The compressibility of the granular fill is represented by a layer of Winkler springs attached to the bottom of the Pasternak shear layer. The membrane divides the shear layer into two parts, one above and the other below the membrane. The general assumptions are that the geosynthetic reinforcement is linearly elastic, rough enough to prevent slippage at the interface with soil and has no shear resistance. A rigid-perfectly plastic friction model is adopted to represent the behaviour of the soil-geosynthetic interface in shear. The saturated soft foundation soil is idealised by the Terzaghi consolidation model which has dashpots and springs. The spring represents the soil skeleton and the dashpot simulates the dissipation of excess pore water pressure of the soil. The spring constant is assumed to have a constant value with depth of the foundation soil and also with time. The consolidation characteristics of the soil both within the loaded region and beyond it are considered to be the same and their variation in the z-direction is not considered. It is assumed that due to one-dimensional consolidation of the soft soil, the displacement of the membrane is zero at the instant at which the load is applied and the deformation takes place only after a finite time has elapsed. The equation governing the response of the proposed model at any particular instant of time ($t>0$) is derived by considering the equilibrium of forces on different elements of the shear layer and the stretched rough elastic membrane at that instant (Fig. 4.2).

The vertical force equilibrium equation of the upper shear layer element at time $t>0$ (Fig. 4.2 (a)), can be written as:

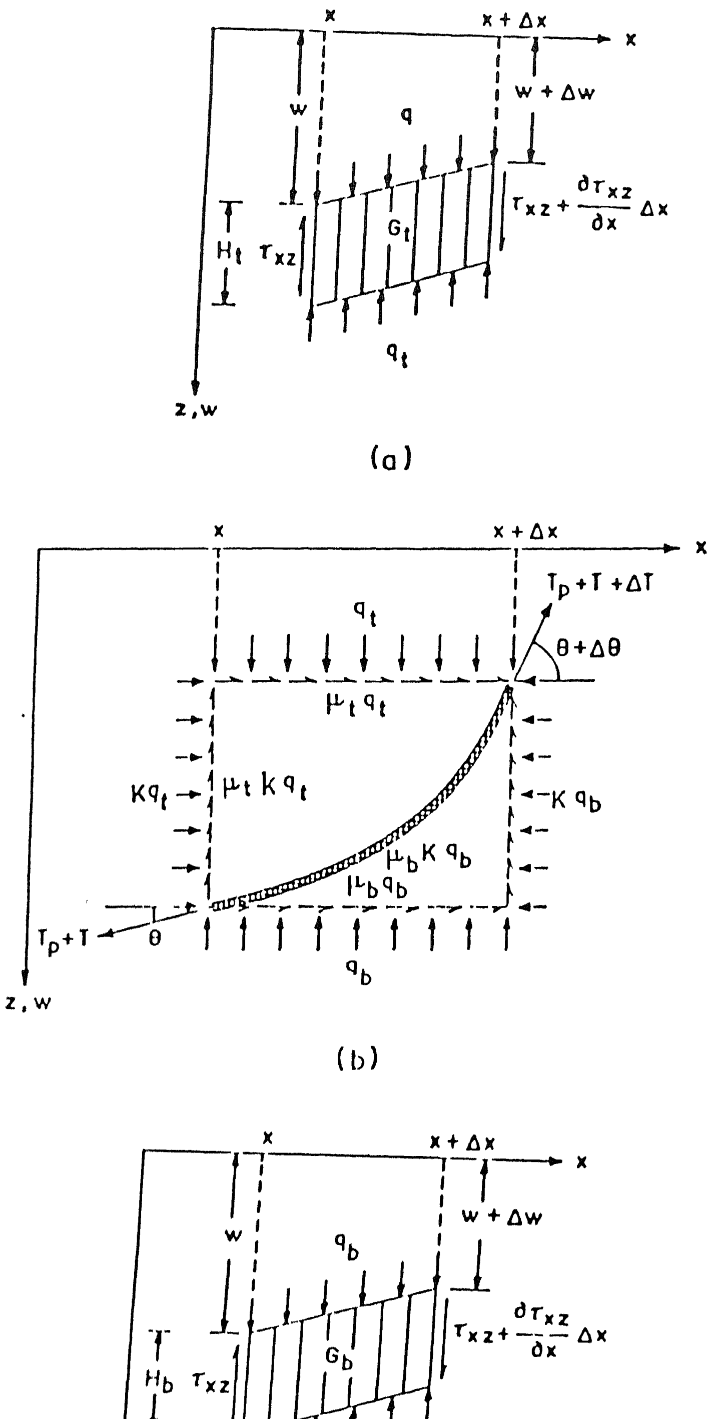


Fig. 4.2. Definition sketch: (a) forces on the upper shear layer element; (b) forces on the membrane element; (c) forces on lower shear layer element.

$$q = q_t - G_t H_t \frac{\partial^2 w(x, t)}{\partial x^2} \tag{4.1}$$

where, q is the applied load intensity, q_t is the vertical force interaction between the membrane and the upper shear layer, w(x, t) is the vertical surface displacement, G_t and H_t are respectively the shear modulus and the thickness of the upper shear layer, x is the distance measured from the centre of the loaded region along x-axis, and t is any particular instant of time measured from the instant of loading.

The horizontal force equilibrium equation of the stretched rough elastic membrane element at time t>0 (Fig. 4.2 (b)), can be written as:

$$(T_p + T + \Delta T)\cos(\theta + \Delta\theta) - (T_p + T)\cos\theta + (\mu_t q_t + \mu_b q_b)\Delta x +$$

$$K(q_t - q_b)\Delta x \tan\theta = 0 \tag{4.2}$$

where, q_b is the vertical force interaction between the membrane and the lower shear layer, μ_t and μ_b are respectively the interface friction coefficients at the top and bottom faces of the membrane, K is the coefficient of lateral stress as defined in Chapter 3, θ is the slope of the membrane, T(x, t) is the tensile force per unit length mobilised in the membrane, and T_p is the pretension per unit length applied to the membrane.

As $\Delta x \to 0$, eqn (4.2) reduces to:

$$\cos\theta \frac{\partial T(x, t)}{\partial x} - (T_p + T)\sin\theta \frac{\partial\theta}{\partial x} = -(\mu_t q_t + \mu_b q_b) - K(q_t - q_b)\tan\theta \tag{4.3}$$

The vertical force equilibrium equation for the stretched rough elastic membrane element at time t>0 (Fig. 4.2 (b)), can be written as:

$$(T_p + T + \Delta T)\sin(\theta + \Delta\theta) - (T_p + T)\sin\theta - (q_t - q_b)\Delta x +$$

$$K(\mu_t q_t + \mu_b q_b)\Delta x \tan\theta = 0 \tag{4.4}$$

As $\Delta x \to 0$, eqn (4.4) reduces to:

$$\sin\theta \frac{\partial T(x,t)}{\partial x} + (T_p + T)\cos\theta \frac{\partial\theta}{\partial x} = (q_t - q_b) - K(\mu_t q_t + \mu_b q_b)\tan\theta \tag{4.5}$$

Using eqns (4.3) and (4.5), one can write:

$$q_t = q_b + \frac{(T_p + T)\sec\theta}{1 + K\tan^2\theta}\frac{\partial\theta}{\partial x} - \frac{(1-K)(\mu_t q_t + \mu_b q_b)\tan\theta}{1 + K\tan^2\theta} \tag{4.6}$$

Substituting for dθ/dx in terms of the vertical surface displacement, w(x, t), into eqn (4.6) one can write:

$$q_t = \bar{X}_1 q_b - \bar{X}_2 (T_p + T)\cos\theta \frac{\partial^2 w}{\partial x^2} \tag{4.7}$$

where,

$$\bar{X}_1 = \frac{1 + K\tan^2\theta - (1-K)\mu_b\tan\theta}{1 + K\tan^2\theta + (1-K)\mu_t\tan\theta} \tag{4.8a}$$

and

$$\bar{X}_2 = \frac{1}{1+K\tan^2\theta+(1-K)\mu_f\tan\theta} \tag{4.8b}$$

The vertical force equilibrium equation of the lower shear layer element at time, t>0, (Fig. 2(c)) can be written as:

$$q_b = q_s - G_b H_b \frac{\partial^2 w(x,t)}{\partial x^2} \tag{4.9}$$

where q_s is the vertical force interaction between the lower shear layer and the saturated soft foundation soil, G_b and H_b are respectively the shear modulus and the thickness of the lower shear layer.

The expression for q_s at any time, t>0, is given by:

$$q_s = k_f w_1 \tag{4.10}$$

where, w_1 (= w - w_2) is the deflection of the spring layer attached directly to the bottom of the Pasternak shear layer representing the compressibility of the granular fill, w_2 is the deflection of spring-dashpot layer representing the soft foundation soil and k_f. is the spring constant per unit area for springs attached to the bottom of the Pasternak shear layer representing the compressibility of the granular fill.

The expression for q_s at time, t>0, can also be given by using Terzaghi effective stress principle (Terzaghi, 1943) as:

$$q_s = \sigma' + u_e \tag{4.11}$$

where, σ' and u_e are respectively the effective stress and the average excess pore water pressure at time, t, in the spring-dashpot system. The effective stress, σ', at time, t>0, is given by:

$$\sigma' = k_s w_2(x,\ t) \tag{4.12}$$

where, k_s is the spring constant per unit area for the spring with the dashpot. The average excess pore water pressure, u_e, at time, t>0, is given by:

$$u_e \;=\; \sum_{m=0}^{m=\infty} \frac{2u_0}{M^2}\, e^{-M^2 T_v} \tag{4.13}$$

in which u_0 is the initial excess pore water pressure; $M = (2m+1)\pi/2$, m= 0, 1, 2, 3,.....; T_v ($= C_v\, t/H_s^2$) is the time factor for primary consolidation, C_v is the average coefficient of consolidation, H_s is the thickness of the soft foundation soil. The eqn (4.13) is the well known analytical solution of the Terzaghi consolidation equation for saturated soft foundation soil with boundaries as shown in Fig. 4.1 and for a uniform initial excess pore pressure distribution (Terzaghi, 1943).

Combining eqns (4.1) and (4.5) - (4.11), one can write:

$$q = \overline{X}_1 \frac{k_f k_s\, w}{k_s + k_f\, U} - \{G_t H_t + \overline{X}_2 (T_p + T)\cos\theta + \overline{X}_1 G_b H_b\} \frac{\partial^2 w}{\partial x^2} \tag{4.14}$$

where, U is the degree of consolidation given as:

$$U = 1 - \sum_{m=0}^{m\to\infty} \frac{2}{M^2} e^{-M^2 T_v} \tag{4.15}$$

The expression for the variation of the mobilised tension in the membrane at any time t>0 is determined by combining eqns (4.1), (4.3), (4.5) and (4.9) - (4.13) and is given by:

$$\frac{\partial T}{\partial x} = -\overline{X}_3 (q + G_t H_t \frac{\partial^2 w}{\partial x^2}) - \overline{X}_4 (\frac{k_f k_s w}{k_s + k_f U} - G_b H_b \frac{\partial^2 w}{\partial x^2}) \tag{4.16}$$

where,

$$\overline{X}_3 = \mu_f \cos\theta (1 + K \tan^2\theta) - (1 - K)\sin\theta \tag{4.17a}$$

and

$$\overline{X}_4 = \mu_b \cos\theta (1 + K \tan^2\theta) + (1 - K)\sin\theta \tag{4.17b}$$

Equations (4.14) and (4.16) govern the response of the proposed foundation model. It can be observed that for the particular values of the parameters, these equations govern the response of the existing foundation models for geosynthetic-reinforced soil (Madhav & Poorooshasb, 1988; Ghosh, 1991) and also for general soil behaviour (Winkler, 1867; Filonenko-Borodich, 1940; Pasternak, 1954). For example, for no prestressing (T_p = 0), and for very small deformation of the geosynthetic reinforcement ($\tan\theta$ = 0) along with α = infinity and U = 1, the present model reduces to the model suggested by Madhav and Poorooshasb (1988). The model suggested by Ghosh (1991)

results in, for $T_p = 0$, $\alpha = 1$, $U = 1$ and $K = 0$, that is, when only the horizontal shear stress transfer is considered at the fill-geosynthetic interface.

The parameters k_f, k_s, G_t and G_b can be determined along the lines suggested by Selvadurai (1979) for mechanical foundation models. The friction coefficients, μ_t and μ_b, at the fill-geosynthetic interface can be determined by the procedures described by Hausmann (1990), and Koerner (1990). The value of K can be determined as described in Chapter 3.

4.3 METHOD OF SOLUTION

To observe the settlement response of the proposed model, eqns (4.14) and (4.16) are stated in their nondimensional forms as:

$$q^* = \overline{X}_1 \frac{\alpha W}{1+\alpha U} - \{G_t^* + \overline{X}_2 (T_p^* + T^*)\cos\theta + \overline{X}_1 G_b^*\} \frac{\partial^2 W}{\partial X^2} \tag{4.18}$$

and

$$\frac{\partial T^*}{\partial X} + \overline{X}_3 \left(q^* + G_t^* \frac{\partial^2 W}{\partial X^2}\right) + \overline{X}_4 \left(\frac{\alpha W}{1+\alpha U} - G_b^* \frac{\partial^2 W}{\partial X^2}\right) = 0 \tag{4.19}$$

where, $X = x/B$, $W = w/B$, $G_t^* = G_t H_t / k_s B^2$, $G_b^* = G_b H_b / k_s B^2$, $q^* = q/k_s B$, $T_p^* = T_p/k_s B^2$, and $T^* = T/k_s B^2$, α $(= k_f / k_s)$ is the spring constant ratio and B is half width of the strip loading.

Writing eqns (4.18) and (4.19) in finite difference form within the specified time-space domain, for an interior node, (i, j), where i and j are indices for space and time respectively, one gets

$$q_i^* = \overline{X}_{1i,j}\,\frac{\alpha W_{i,j}}{1+\alpha U_j} - \{G_t^* + \overline{X}_{2i,j}\,(T_p^* + T_{i,j}^*)\cos\theta_{i,j} + \overline{X}_{1i,j}\,G_b^*\}\,\mathrm{x}$$

$$\{\frac{W_{i-1,j} - 2W_{i,j} + W_{i+1,j}}{(\Delta X)^2}\} \tag{4.20}$$

and

$$T_{i,j}^* = T_{i+1,j}^* + (\Delta X/4)[(\overline{X}_{3i,j} + \overline{X}_{3i+1,j})\,\{(q_i^* + q_{i+1}^*) + G_t^*(\frac{\partial^2 W}{\partial X^2}|_{i,j} + \frac{\partial^2 W}{\partial X^2}|_{i+1,j})\}$$

$$+ (\overline{X}_{4i,j} + \overline{X}_{4i+1,j})\,\{\frac{\alpha(W_{i,j} + W_{i+1,j})}{1+\alpha U_j} - G_b^*(\frac{\partial^2 W}{\partial X^2}|_{i,j} + \frac{\partial^2 W}{\partial X^2}|_{i+1,j})\}] \tag{4.21}$$

It may be noted that the applied load, q^*, carries a subscript, i, only as the applied load does not vary with time in the present study. Similarly, the degree of consolidation, U, is being taken as a function of time alone and hence it carries a subscript, j, only. In eqn (4.21), the derivative $\frac{d^2W}{dX^2}$ has been expressed by central finite difference scheme while $\frac{dT^*}{dX}$ has been expressed by forward finite difference scheme. Hence, in order to minimise the numerical error, average values of q^*, W, $\frac{\partial^2 W}{\partial X^2}$, $\overline{X}_3$ and $\overline{X}_4$, for each element, have been taken in eqn (4.21).

The solutions are obtained for a uniform nondimensional load intensity, q_0^* acting over a width 2B (Fig. 4.3). Since there is a symmetry about the centre of the loaded region, the

slope of the settlement-distance profile at the centre of the loaded region is taken as zero. At the edge of the reinforced zone (i.e. at X = L/B, L being half width of reinforced zone), the slope is considered as zero as observed in most of the practical cases whether the membrane is free or fixed. The mobilised tensile force at the edge of the reinforcement is considered as zero (i. e. $T^* = 0$ for X = L/B), which implies that the frictional resistance mobilised over the length of the membrane is sufficient to balance tensile forces.

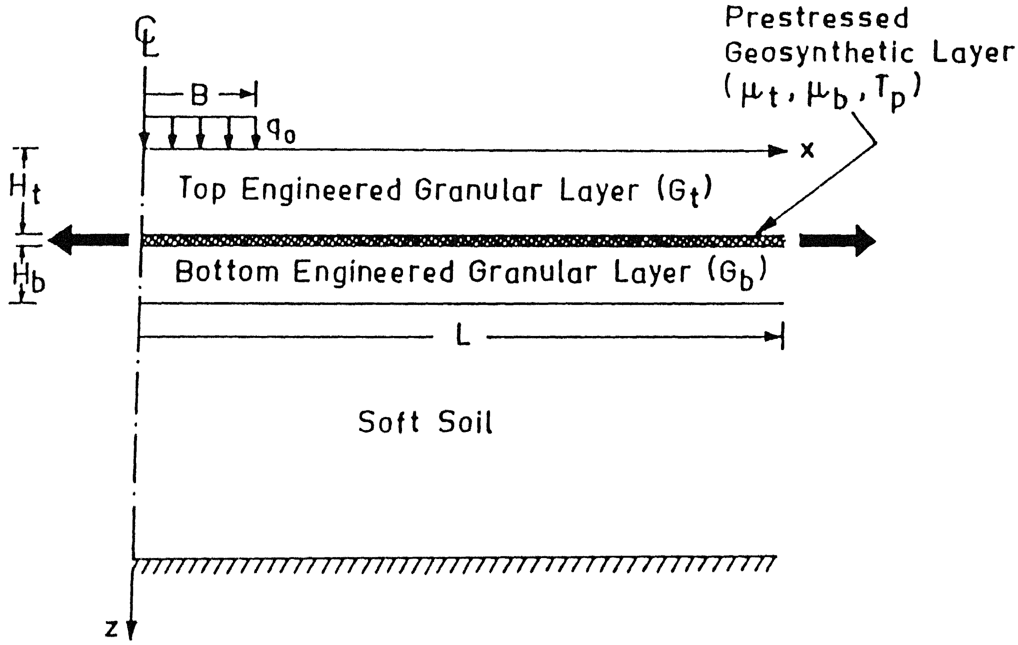


Fig. 4.3 Definition sketch: foundation and loading systems analysed.

Based on the above formulation, results have been obtained using the HP-9000/850 computer system. Due to the symmetry of the problem analysed, only half region of the problem (Fig. 4.3) is considered. The solutions have been obtained with a convergence criterion as (Chapra and Canale, 1989):

$$\left| \frac{W_i^k - W_i^{k-1}}{W_i^k} \right| X\,100\% < \varepsilon_s \qquad (4.22)$$

for all i, where k and k -1 are, respectively, the present and previous iterations, and ε_s is the specified tolerance which has been considered to be 0.0001 in the present study. The ranges of various nondimensional parameters studied are shown in Table 1.

TABLE 4.1

Ranges of Nondimensional Parameters Studied

Sl. No.	Nondimensional Parameters	Ranges
1.	Load intensity, q^*	0.01 - 1.0
2.	Shear parameter, G_t^* or G_b^*	0.01 - 1.0
3.	Interface friction coefficient, μ_t or μ_b	0.1 - 1.0
4.	Width of reinforced zone, L/B	1.0 - 3.0
5.	Lateral stress ratio, K	0.4 - 1.0
6.	Prestress in the geosynthetic reinforcement, T_p^*	0.0-1.0
7.	Modular ratio, α (= k_f/k_s)	5 - infinity
8.	Degree of consolidation, U	0 - 100%

4.4 COMPARISON WITH EXISTING MECHANICAL MODELS

The settlement predictions obtained by the present mechanical foundation model are compared with those computed using the mechanical foundation models, suggested in recent past by Madhav and Poorooshasb (1988), and Ghosh (1991). For common model parameters, it is tried to observe the effect of more general shear stress transfer mechanism at the fill-geosynthetic interface, considered by the present model. Throughout the

comparison, $K = K_0 = 0.4$, $T_p^* = 0$, $\alpha = \infty$, $U = 100\%$ are considered in the present model to make the comparison under similar situations.

Figure 4.4 shows the typical settlement profiles of geosynthetic-reinforced granular fill - soft soil system obtained by the present model for various nondimensional load intensities. The settlement profiles, as obtained by using the models suggested by Madhav and Poorooshasb, and Ghosh, are plotted for the sake of comparison. It is observed that at any location, the settlement obtained by the present model is more than the settlement obtained by the Madhav-Poorooshasb model, but is less than the settlement predicted by the Ghosh model. It can also be noticed that the present predictions of settlement differ much from those obtained by the existing models at a higher

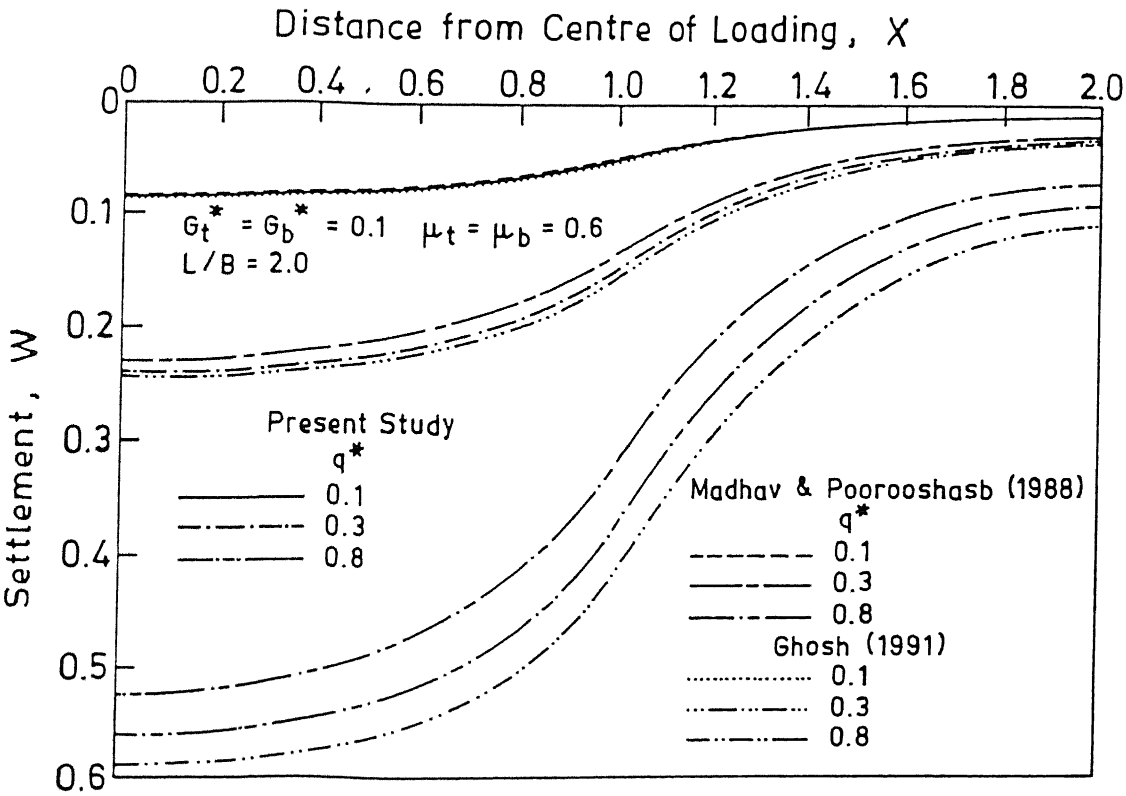


Fig. 4.4. Settlement profiles - comparison of foundation models for various load intensities.

nondimensional load intensity, but the difference is less at a lower value of load intensity. For example, at the centre of the loaded region, the settlement obtained by the present model is more by 3.91% for $q^* = 0.3$ and by 7.25% for $q^* = 0.8$ than the settlement obtained by Madhav-Poorooshasb model whereas, the present value is less by 2.05% for $q^* = 0.3$ and by 4.58% for $q^* = 0.8$ than the settlement predicted by Ghosh model. At the edge of the loaded region, the present value of settlement obtained is more by 9.63% for $q^* = 0.3$ and by 17.40% for $q^* = 0.8$ than the settlement obtained by Madhav-Poorooshasb model whereas, the present value is less by 5.13% for $q^* = 0.3$ and by 9.78% for $q^* = 0.8$ than the settlement obtained by Ghosh model. For $q^* = 0.1$, the settlements obtained by the present model are almost same as those obtained by Madhav-Poorooshasb, and Ghosh models.

Figure 4.5 illustrates the comparison of typical mobilised tensile force distribution profiles in the geosynthetic reinforcement obtained by the present model with those obtained by using the models suggested by Madhav and Poorooshasb, and Ghosh for several load intensities. It is observed that the predictions differ significantly at higher nondimensional load intensities. At the centre of the loaded region, the mobilised tensile force obtained by the present model is more by 4.79% for $q^* = 0.3$ and by 12.16% for $q^* = 0.8$ than the value obtained by Madhav-Poorooshasb model whereas, the present value of settlement is less by 1.96% for $q^* = 0.3$ and by 4.07% for $q^* = 0.8$ than the value obtained by Ghosh model. For $q^* = 0.1$, the predictions are almost the same.

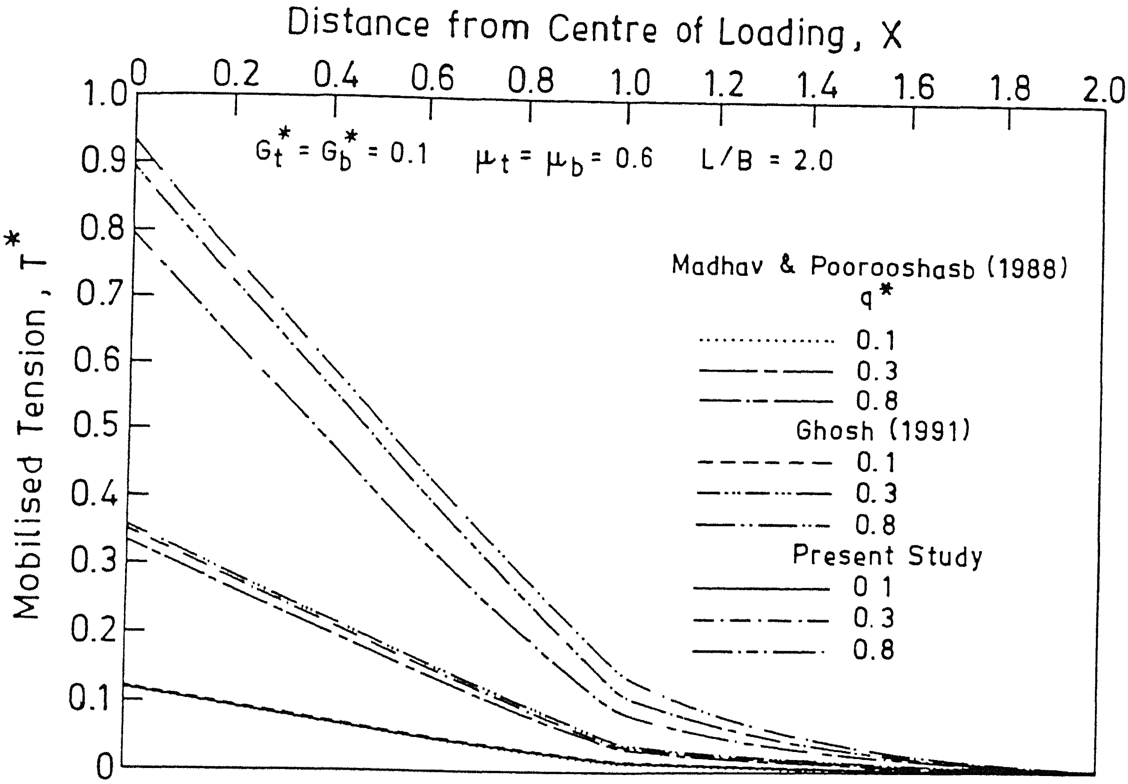


Fig. 4.5. Mobilised tensile force distribution profiles - comparison of foundation models for various load intensities.

The comparison of typical settlement profiles for several shear parameters of granular fill are shown in Fig. 4.6. It is noted that for lower values of shear parameter, the differences in values of settlement obtained by the present model and the models suggested by Madhav and Poorooshasb, and Ghosh are large at any location. The settlement obtained by the present model at any location is more than the value obtained by Madhav-Poorooshasb model but, is less than that obtained by Ghosh model. For example, at the centre of the loaded region, for $G_t^* = G_b^* = 0.1$, the settlement obtained by

the present model is more by 7.25% than the settlement obtained by Madhav-Poorooshasb model but, is less by 4.58% than the settlement obtained by Ghosh model.

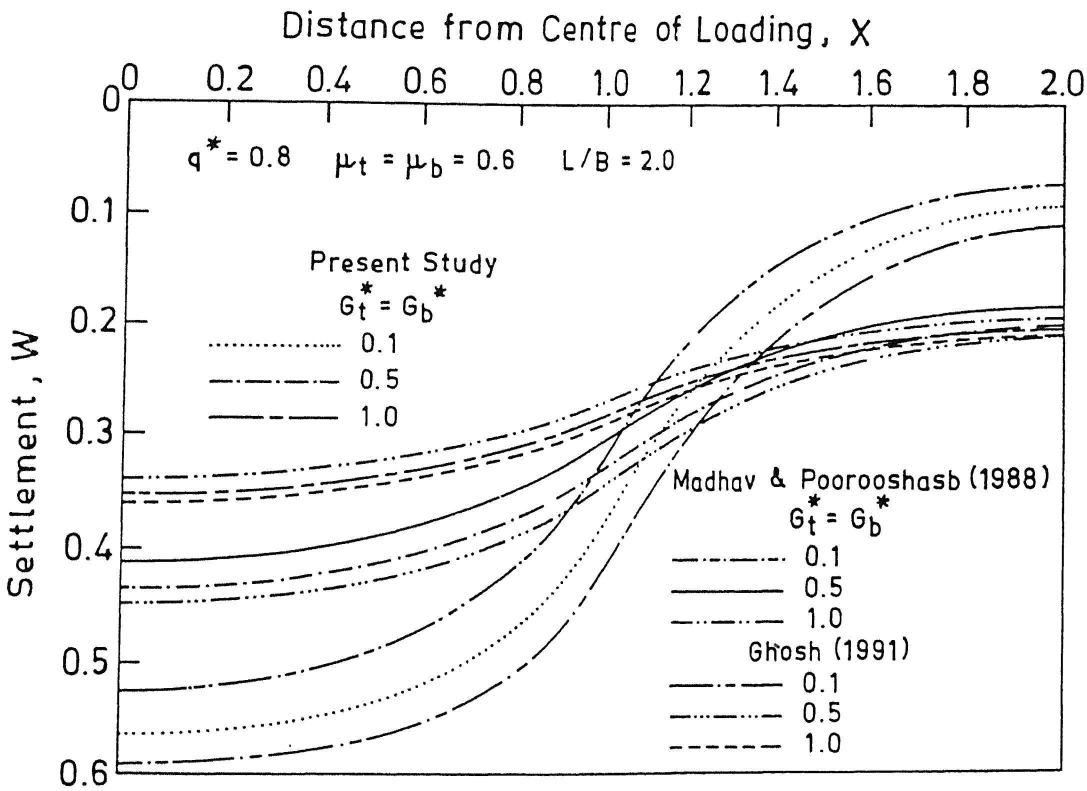


Fig. 4.6. Settlement profiles - comparison of foundation models for various shear parameters of the granular fill.

The comparison of typical settlement profiles for various interfacial friction coefficients at the fill-geosynthetic interface are shown in Fig. 4.7. It is noticed that the settlements obtained by the present model and the models suggested by Madhav and Poorooshasb, and Ghosh differ significantly at higher values of interfacial friction coefficient. For $\mu_t = \mu_b = 1.0$, the settlement obtained by the present model at the centre of the

loaded region is more by 11.65% than the value obtained by Madhav-Poorooshasb model but, is less by 6.27% than the value obtained by Ghosh model.

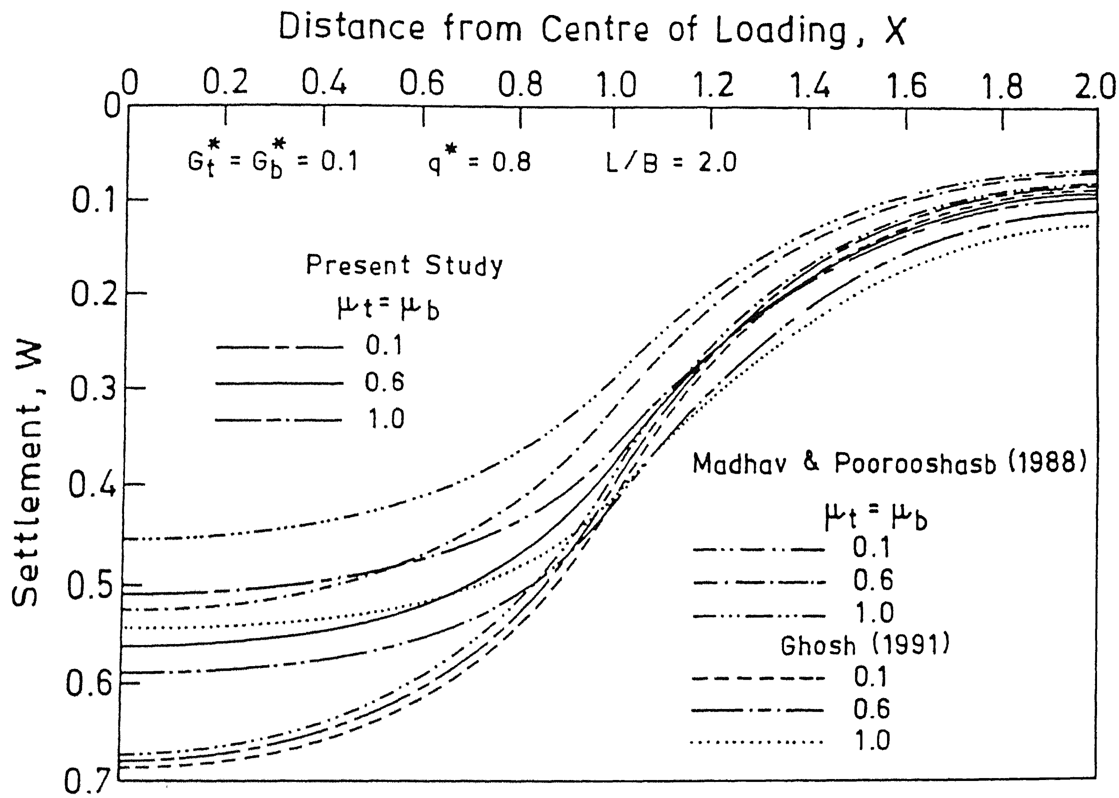


Fig. 4.7. Settlement profiles - comparison of foundation models for various interfacial friction coefficients.

Table 4.2 shows the settlements obtained by the present model at the centre and at the edge of the loaded region for a particular set of parameters for various widths of reinforced zone. The settlements computed using models suggested by Madhav and Poorooshasb, and Ghosh are also presented in the table for the sake of comparison. It is observed that the present values of settlement lie between the values computed using the models suggested by Madhav and Poorooshasb, and Ghosh. Further, it is noted that the results differ much for higher width of reinforced zone.

TABLE 4.2

Comparison of Foundation Models for Different Widths of Reinforced Zone

$q^* = 0.8 \quad G_b^* = G_b^* = 0.1 \quad \mu_t = \mu_b = 0.6$						
L/B	Madhav & Poorooshasb (1988)		Ghosh (1991)		Present Study	
L/B	1.2	2.0	1.2	2.0	1.2	2.0
W_c /B	0.570	0.524	0.618	0.589	0.602	0.562
W_c /B	0.460	0.322	0.540	0.419	0.512	0.378

The comparison of results obtained by the present model for various parameters with the results obtained by the models suggested by Madhav and Poorooshasb (1988), and Ghosh (1991) reveals that the present results are always more than those obtained by Madhav-Poorooshasb model and are less than those obtained by Ghosh model, especially in cases of large settlements. The model proposed by Madhav and Poorooshasb is applicable for problems of small deformations. The model proposed by Ghosh considers only horizontal shear stress transfer at the fill-geosynthetic interface and large deformations of the membrane, which predicts higher results than expected. This is mainly because the vertical shear stress transfer at the fill-geosynthetic interface becomes significant at large settlements. The present model considers these aspects appropriately.

4.5 DISCUSSION OF RESULTS

4.5.1 Effect of Lateral Stress Ratio

In this section, the results obtained for various lateral stress ratios are presented. For a given granular fill, i.e. for a particular value of ϕ', an increase in the lateral stress ratio

indicates the increase in the horizontal stress which may be induced due to higher compaction effort applied to the soil before the structure is placed. Throughout the study on the effect of lateral stress ratio in granular fill on the settlement characteristics of geosynthetic-reinforced granular fill - soft soil system, $T_p^* = 0.0$, $\alpha = 1$, and $U = 100\%$ are considered in the present model.

Figure 4.8 shows typical load-settlement curves for geosynthetic-reinforced granular fill - soft soil system at the centre and at the edge of the loaded region for various values of lateral stress ratio. It is observed that the increase of the value of K results in significant reductions in settlement both at the centre and at the edge of the loaded region for higher load intensities. At much lower load intensities, appreciable reductions in settlement are not noticed with the increase of K. It is also noted that settlement reduction is more at the edge than that at the centre for any increase of K. For example, as K increases from 0.4 to 1.0, the settlement reduces by 1.19%, 6.13%, and 11.29% at the centre of the loaded region for $q^* = 0.1$, 0.5, and 1.0 respectively whereas, the corresponding settlement reductions at the edge are 4.08%, 13.89%, and 20.37% respectively. Thus, one can reduce the settlement of the geosynthetic-reinforced granular fill - soft soil system significantly by increasing the horizontal stress in the granular fill. It can also be noticed that for higher nondimensional load intensities, the geosynthetic-reinforced granular fill soil system behaves as a much stiffer system in comparison to its behaviour at lower load intensities, which is a common trend for the geosynthetic-reinforced granular fill - soft soil system observed earlier in both experimental

(Drexel University tests, Koerner, 1990; Abduljauwad *et al.*, 1994) and analytical (Bourdeau, 1989) studies.

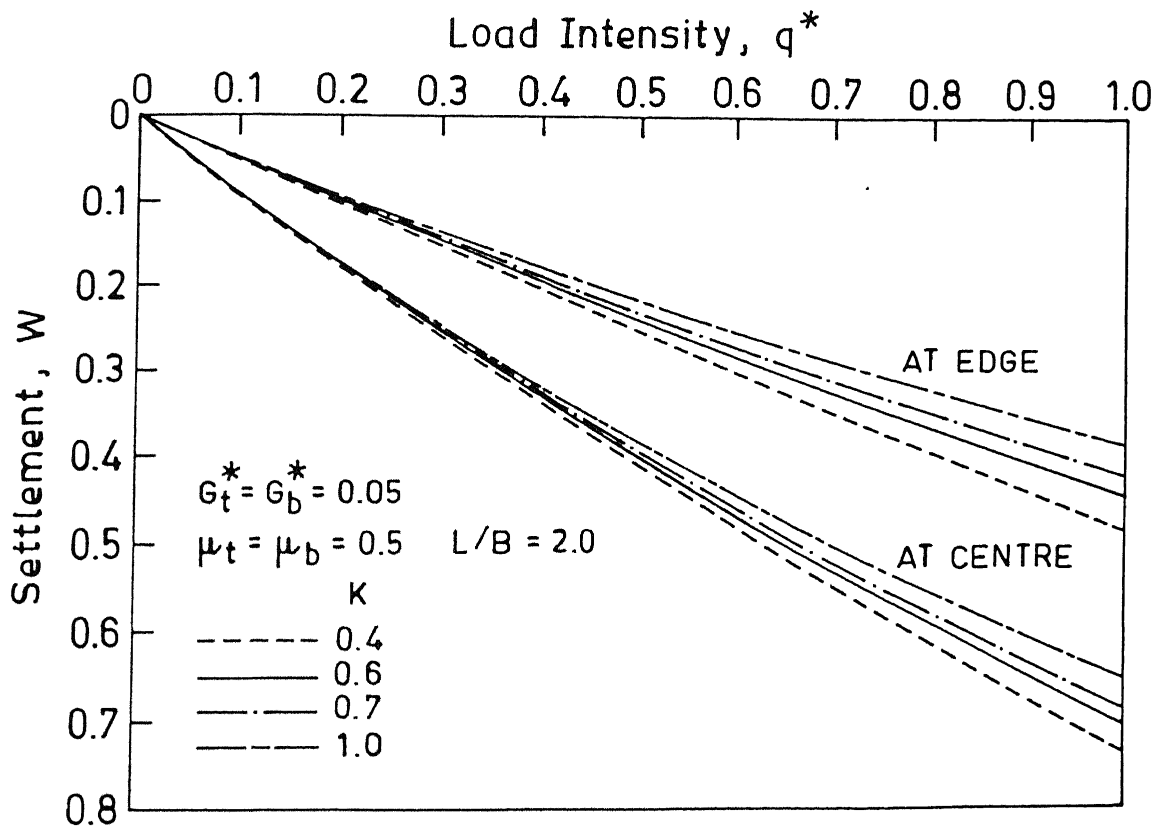


Fig. 4.8. Load-settlement curves - effect of lateral stress ratio.

Figure 4.9 shows typical settlement profiles for different shear parameters of the granular fill, bringing out the effect of lateral stress ratios. It is observed that for lower shear parameter, the settlement at any location decreases significantly as the value of K increases. However, for higher shear parameter, it is noted that the increase of K results in relatively less reduction in settlement. For example, as K increases from 0.4 to 1.0, the settlement at the centre of the loaded region decreases by 6.26% for $G_t^* = G_b^* =$

0.01 and by 3.15% for $G_t^* = G_b^* = 0.5$. The corresponding reductions at the edge of the loaded region are 21.35% and 4.25% respectively.

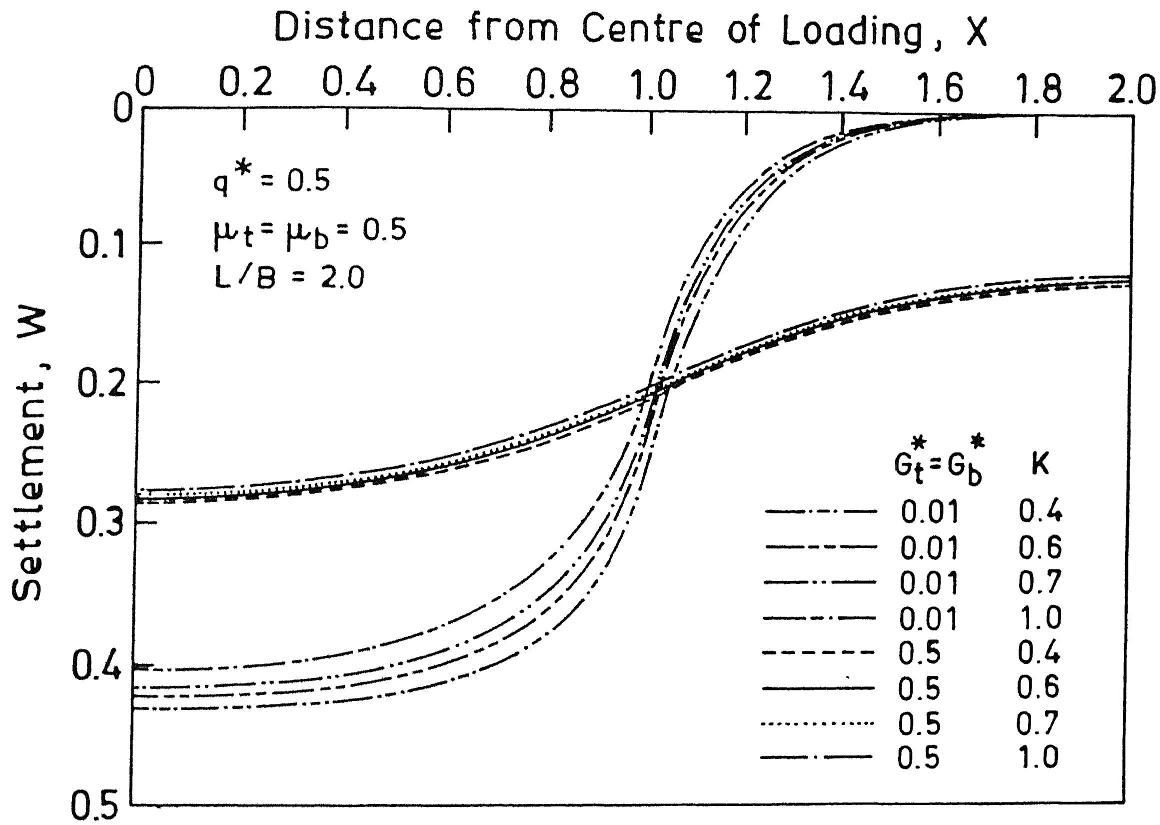


Fig. 4.9. Settlement profiles - effect of lateral stress ratio in the granular fill at its different shear parameters.

Figure 4.10 shows typical settlement profiles for different interfacial friction coefficients, bringing out the effect of lateral stress ratios. It is noticed that the settlement reduces significantly with the increase of K for higher interfacial friction coefficients whereas, for lower interfacial friction coefficients the settlement reductions are relatively less. For example, for the increase of K from 0.4 to 1.0, the settlement at the centre of the loaded region reduces by 1.52% for $\mu_t = \mu_b = 0.1$ and by 10.63% for $\mu_t = \mu_b = 1.0$

whereas, the corresponding settlement reductions at the edge of the loaded region are 3.89% and 21.31% respectively.

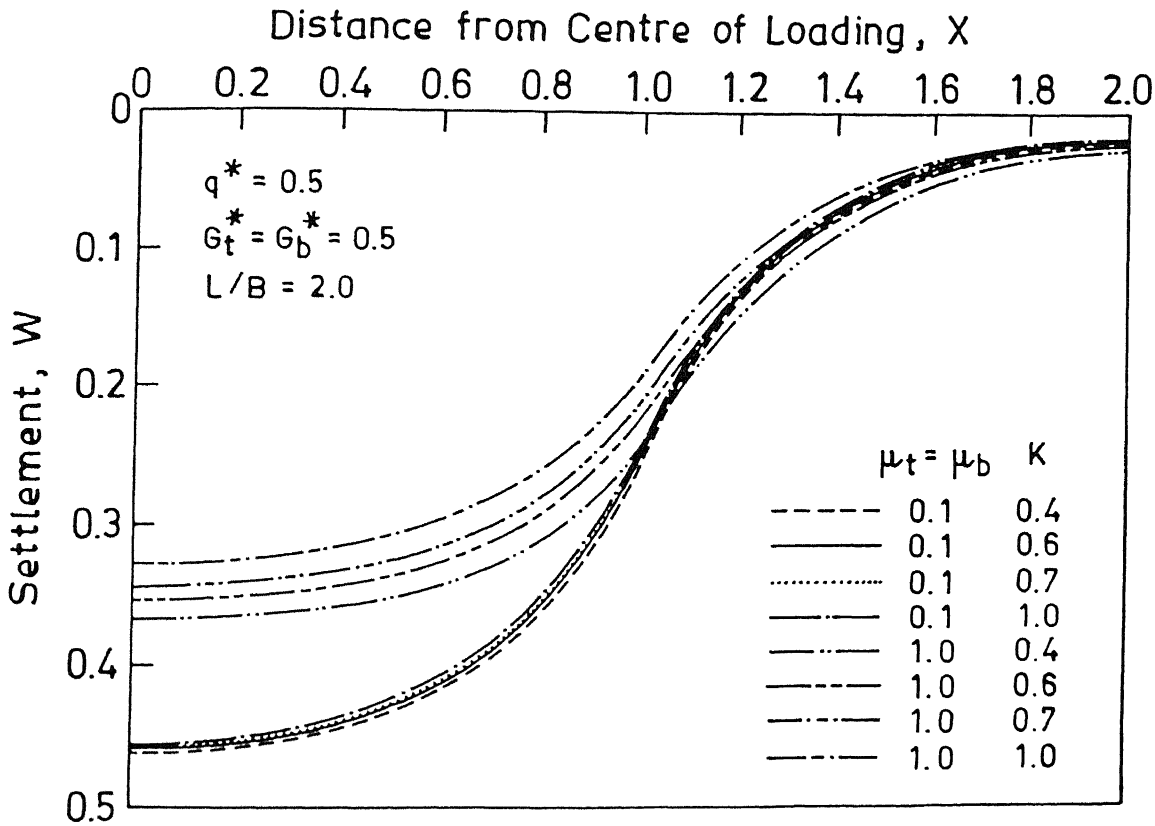


Fig. 4.10. Settlement profiles - effect of lateral stress ratio in the granular fill at different interfacial friction coefficients.

4.5.2 Effect of Prestressing the Geosynthetic Reinforcement

Throughout the study on the effect of prestressing the geosynthetic reinforcement on the settlement characteristics of geosynthetic-reinforced granular fill - soft soil system, $\alpha = 1$, and U = 100% are considered in the present model.

Fig. 4.11 shows typical settlement profiles of a prestressed geosynthetic-reinforced granular fill - soft soil system for various values of prestress in the geosynthetic

reinforcement and simultaneously compares it with reinforced soil system without prestressing of the geosynthetic reinforcement and also with unreinforced soil idealised by Winkler, and Pasternak foundation models. It is observed that the settlement

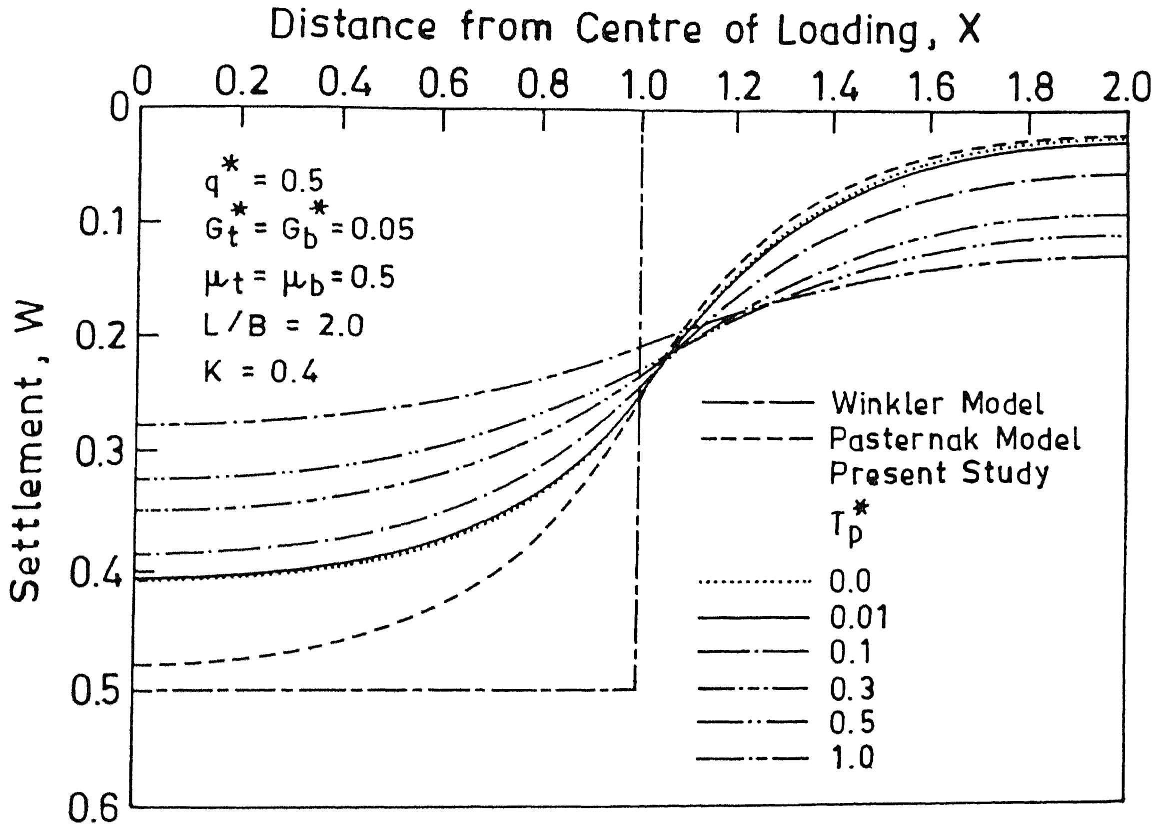


Fig. 4.11. Settlement profiles: effect of prestressing the geosynthetic reinforcement compared with reinforced soil system without prestressing and unreinforced soil.

within the loaded region ($X \leq 0$) reduces as the prestress in the geosynthetic reinforcement is increased whereas, there is an increase in settlement beyond the loaded region. The settlement reductions for nondimensional prestress values of 0.01, 0.1, 0.3, 0.5, and 1.0, compared with no prestress in the geosynthetic reinforcement are respectively 0.74%, 5.64%, 14.22%, 20.59%, and 31.86% at the centre of the loaded region whereas, at the edge the corresponding reductions in settlement are 0.40%, 2.78%, 6.75%, 9.92%,

and 17.06% respectively. These results indicate that the reductions in settlement at the centre of the loaded region due to prestressing are more significant than those at the edge of the loaded region. This fact is more clear from Fig. 4.12 showing typical load-settlement curves. Thus the differential settlement of the loaded region is reduced as a result of prestressing the geosynthetic reinforcement.

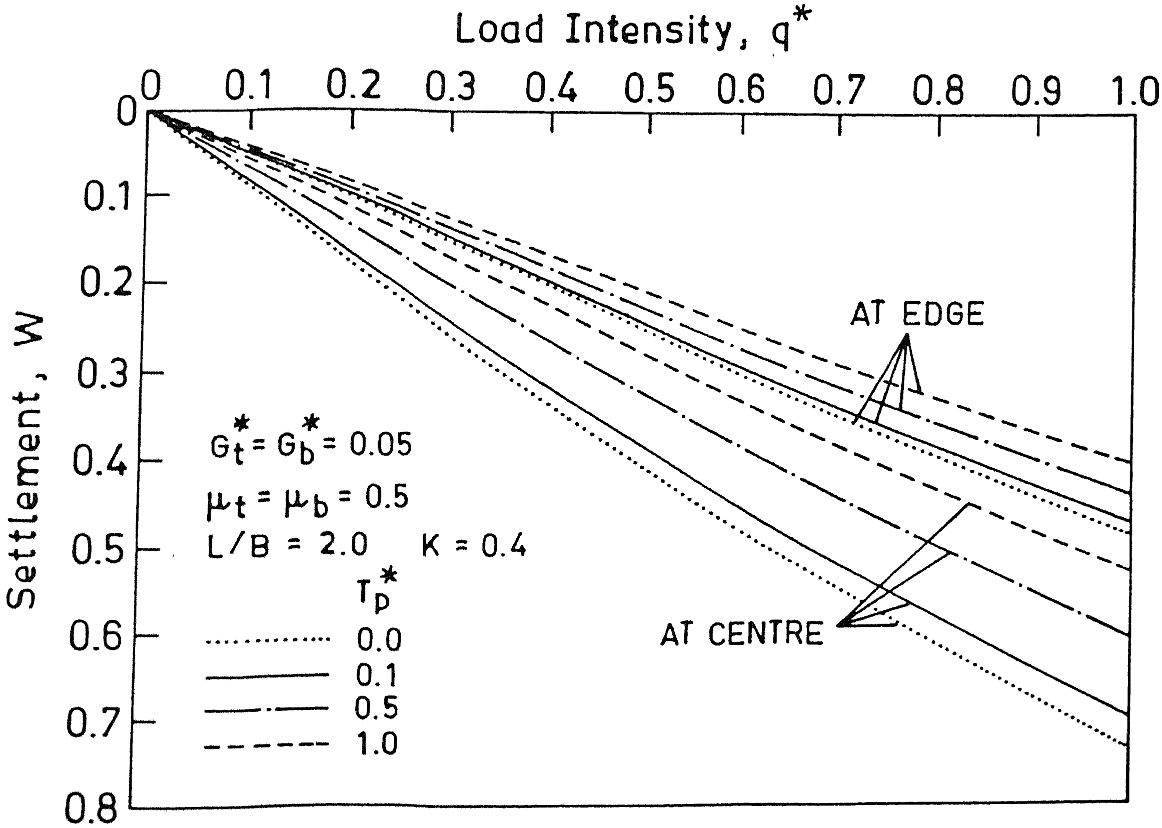


Fig. 4.12. Load-settlement curves for various values of prestress.

From Fig. 4.12, it can also be noted that the order of settlement reduction for any value of prestress in the geosynthetic reinforcement is relatively more for lower load intensities. For example, for $T_p^* = 0.5$, the settlement reduces by 24.30% at the centre of

the loaded region for $q^* = 0.1$ whereas, for $q^* = 1.0$, the corresponding reduction is 17.69%. Hence, it can be stated that to achieve a particular reduction in the settlement, less prestress in the geosynthetic reinforcement is required for lower nondimensional load intensities as compared to higher load intensities. This indicates that in the case of soft foundation soils where relatively more membrane effect of reinforcement is felt for a given load, more prestress is required in the geosynthetic reinforcement to obtain a reduction of particular order in the settlement as compared to stiff foundation soils.

Figure 4.13 shows the effect of prestressing the geosynthetic reinforcement on the typical settlement profiles for different values of nondimensional shear parameter of the granular fill. It is observed that, due to prestress, settlement improves significantly within the loaded region for lower value of shear parameter whereas for higher value of shear parameter, settlement reductions are relatively quite less. For example, for $T_p^* = 0.1$, the settlement reductions are 6.50% and 6.00 % at the centre and at the edge of loaded region respectively for $G_t^* = G_b^* = 0.01$. The corresponding settlement reductions are 1.76% and 1.65% respectively for $G_t^* = G_b^* = 1.0$. This indicates that prestressing the geosynthetic reinforcement might be an effective tool for further reducing settlements where granular fill of low shear modulus or low thickness due to limited availability of granular fill is used.

Figure 4.14 shows typical settlement profiles for different interfacial friction coefficients, bringing out the effect of prestressing the geosynthetic reinforcement. It is noted that the influence of prestressing in settlement reduction is relatively more near

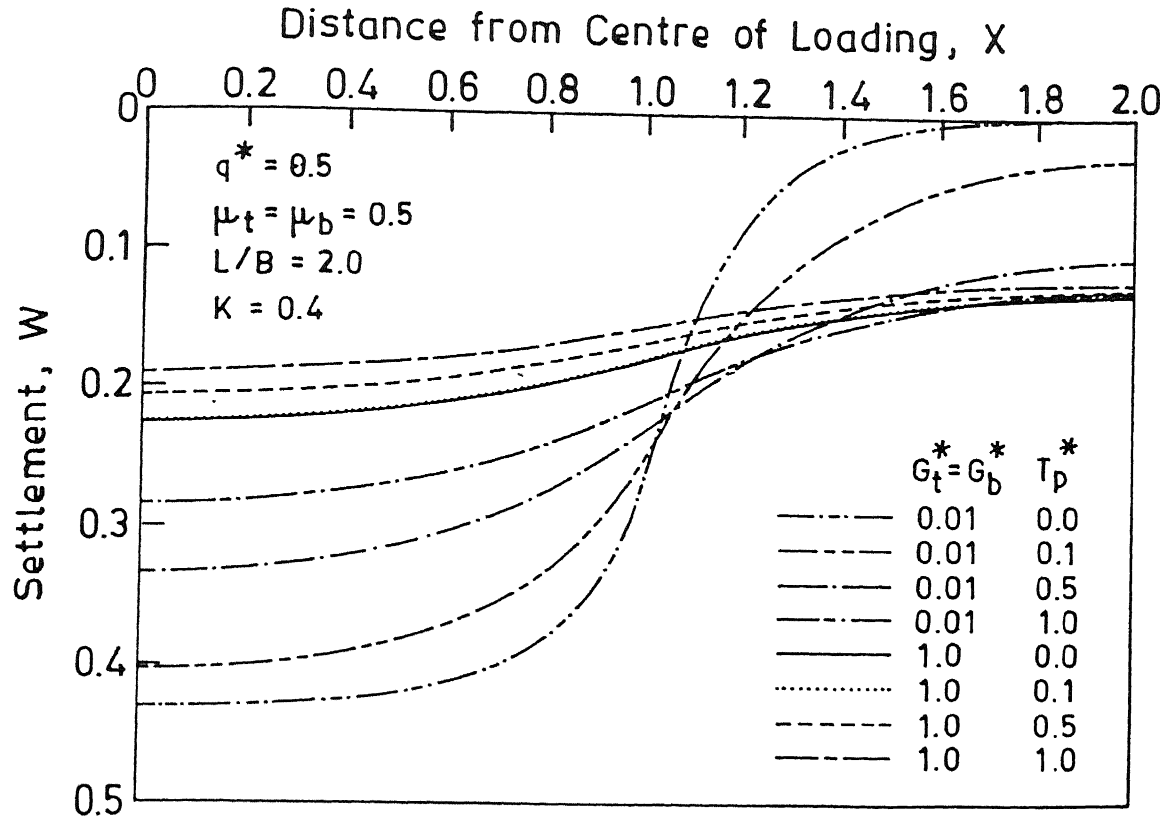


Fig. 4.13. Settlement profiles - effect of prestressing for different shear parameters of granular fill.

the centre of the loaded region but less near the edge for lower value of interfacial friction coefficient. For example, for $T_p^* = 0.5$, the settlement reductions is 24.24% at the centre of the loaded region and is 8.56% at the edge for $\mu_t = \mu_b = 0.1$ whereas, the corresponding settlement reductions are 18.26% and 10.65% respectively for $\mu_t = \mu_b = 1.0$. It appears that, if the geosynthetic layer is placed directly on the soft foundation soil, prestressing will be more effective in settlement reduction, compared to the case when it is placed inside the granular fill, because of the lower interface friction coefficient below the geosynthetic layer in the former case.

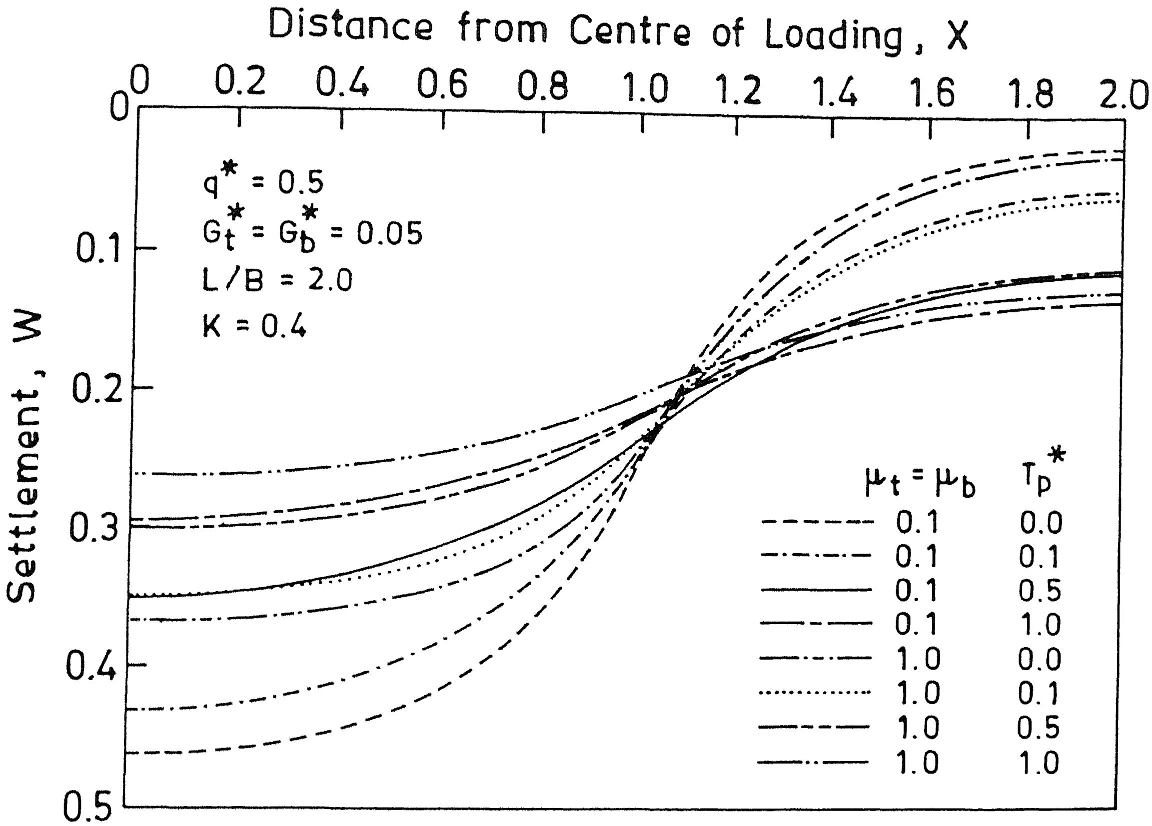


Fig. 4.14. Settlement profiles - effect of prestressing for different interfacial friction coefficients.

Figure 4.15 shows the effect of prestressing on the settlement response of geosynthetic-reinforced soft soil system for different widths of reinforced zone. It is noticed that the improvement in settlement characteristics at any location within the loaded region due to prestress in the geosynthetic reinforcement is relatively more for higher widths of reinforced zone. For example, for $T_p^* = 0.5$, the settlement reduces by 10.92% at the centre and by 2.12% at the edge for L/B = 1.2 whereas, the corresponding reductions are 18.70% and 8.98% respectively for L/B = 2.0.

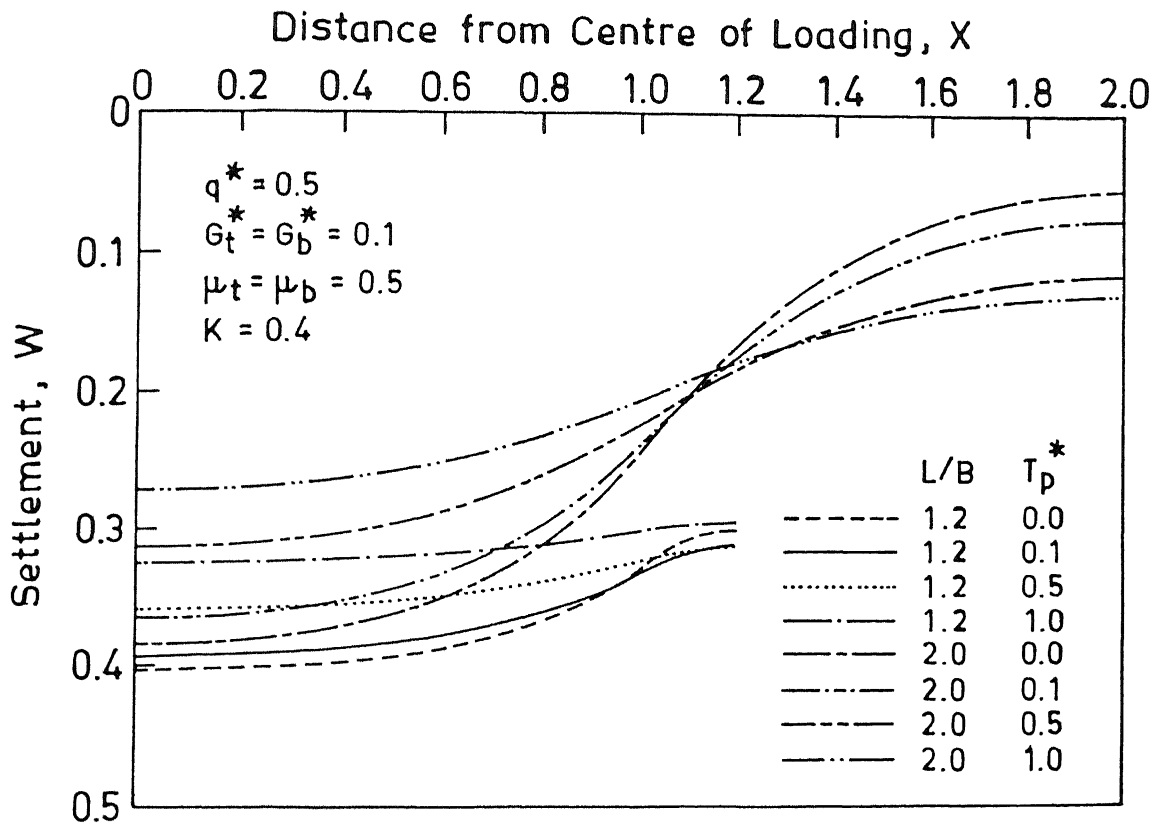


Fig. 4.15. Settlement profiles - effect of prestressing for different widths of reinforced zone.

Figure 4.16 shows typical settlement profiles for different lateral stress ratios in the granular fill, bringing out the effect of prestressing on the settlement response. It is observed that for low prestress, the settlement reduction at any location due to higher horizontal stress in the granular fill is more than the case where high prestress is used. For example, for an increase of K from 0.4 to 1.0, the settlement reduction is 5.19% at the centre of the loaded region and is 11.00% at the edge for $T_p^* = 0.1$. The corresponding values are 2.89% and 4.31% respectively for $T_p^* = 1.0$. These results

indicate that, if the prestressing of geosynthetic is done in geosynthetic-reinforced granular fill - soft soil systems, the effect of horizontal stress in the granular fill is not significant.

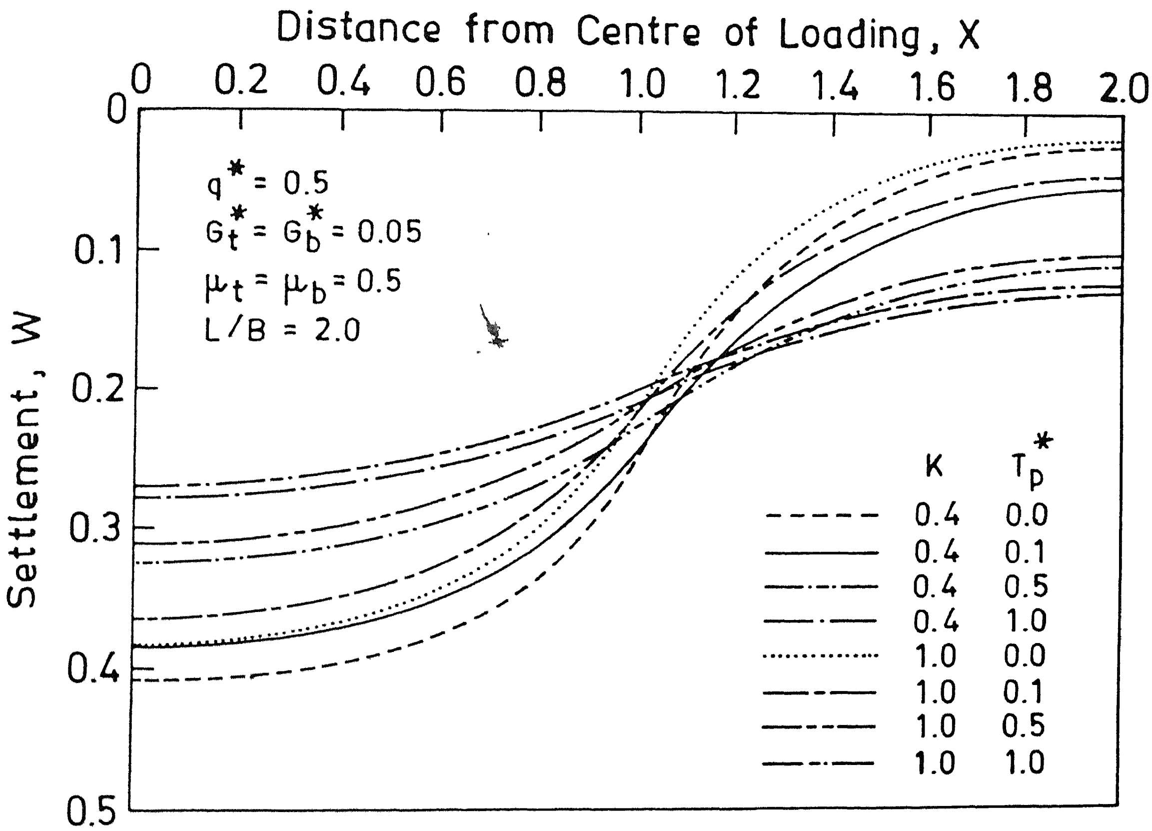


Fig. 4.16. Settlement profiles - effect of prestressing for different lateral stress ratios.

The effect of prestressing on the mobilised tensile force distribution is shown in Fig. 4.17. It is observed that prestress in the geosynthetic reinforcement affects the mobilisation of tensile force in the geosynthetic reinforcement. With the increase in prestress, the mobilised tensile force in the geosynthetic reinforcement increases in most of the locations except near the centre of the loaded region where decrease occurs. The increase of mobilised tensile force is relatively more near the edge of the loaded region.

The mobilised tensile force at X = 2.0 is zero because of one of the boundary conditions assumed.

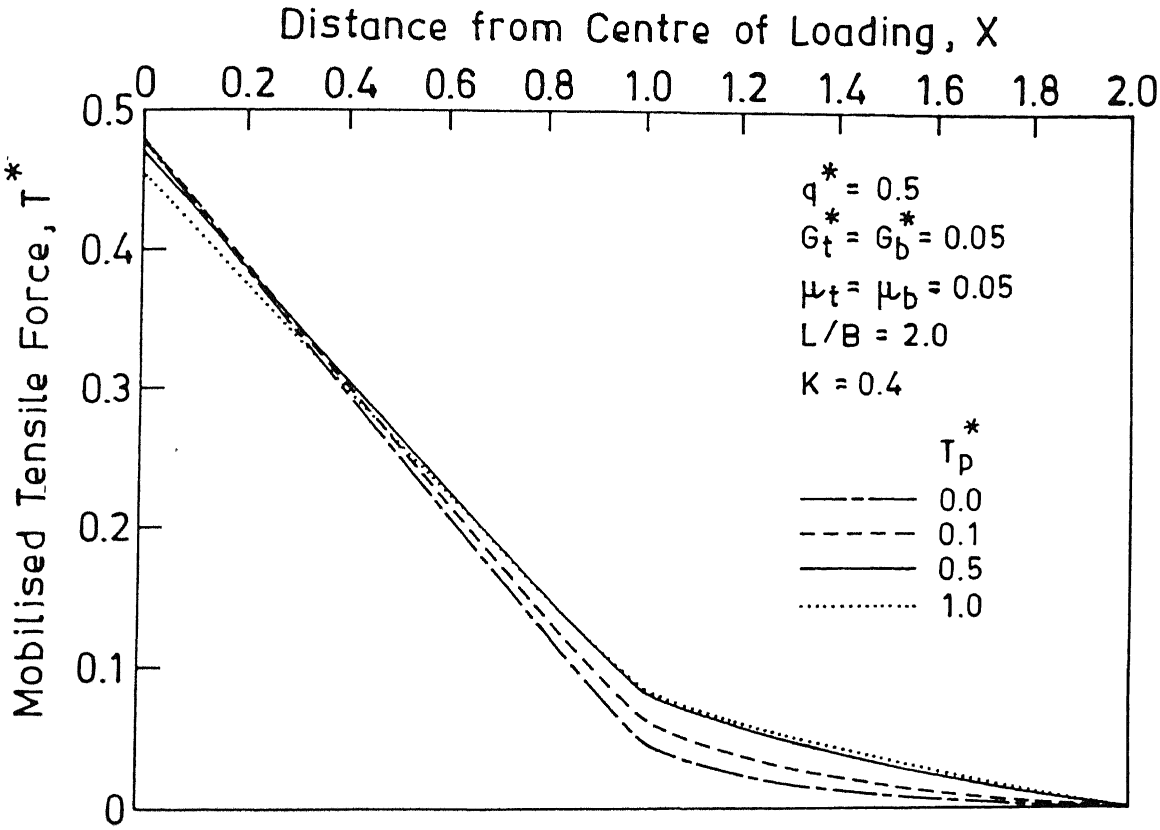


Fig. 4.17. Mobilised tensile force distribution profiles - effect of prestressing the geosynthetic reinforcement.

4.5.3 Effect of Compressibility of the Granular Fill

Throughout the study on the effect of compressibility of the granular fill on the settlement characteristics of geosynthetic-reinforced granular fill - soft soil system, U = 100% are considered in the present model. The modular ratio, α, is the measure of relative compressibility of the granular fill and the soft soil. To study the effect of compressibility of the granular fill, increasing values of α are considered in the present work. As the value

of α increases, the relative compressibility of the granular fill decreases and when modular ratio tends to infinity, the granular fill becomes incompressible.

Figure 4.18 shows the effect of compressibility of the granular fill on the load-settlement characteristics of geosynthetic-reinforced compressible granular fill - soft soil system at the

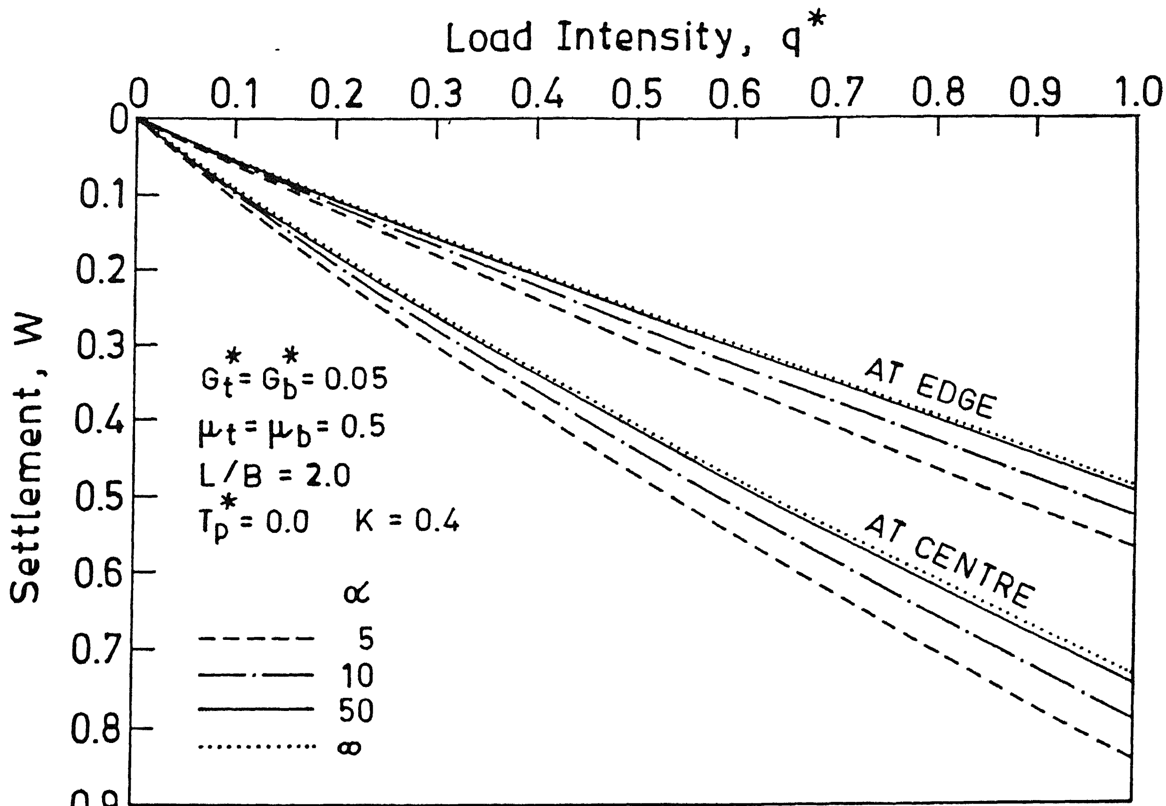


Fig. 4.18. Load-settlement curves - effect of compressibility of granular fill.

centre and at the edge of the loaded region. It is noticed that the order of decrease of the settlement is more at the edge than that at the centre of the loaded region. For example, for $q^* = 1.0$, as α increases from 5 to 50 the settlement reduces by 11.61% at the centre and by 13.33% at the edge of the loaded region. For the increase of α from 5 to ∞, the corresponding reductions are 12.91% and 14.74% respectively. However, for

smaller nondimensional load intensities (less than approximately 0.1), the settlement reductions are very small as the value of α is increased. The comparison of the settlement reductions for the increase of α from 5 to 50 and from 5 to ∞ indicates that when the granular fill is 50 times, or more, stiffer than the soft foundation soil, the compressibility of the granular fill may be ignored in settlement calculations for routine field applications.

Figure 4.19 shows typical settlement profiles for different shear parameters of the granular fill, bringing out the effect of compressibility of the granular fill. It is observed

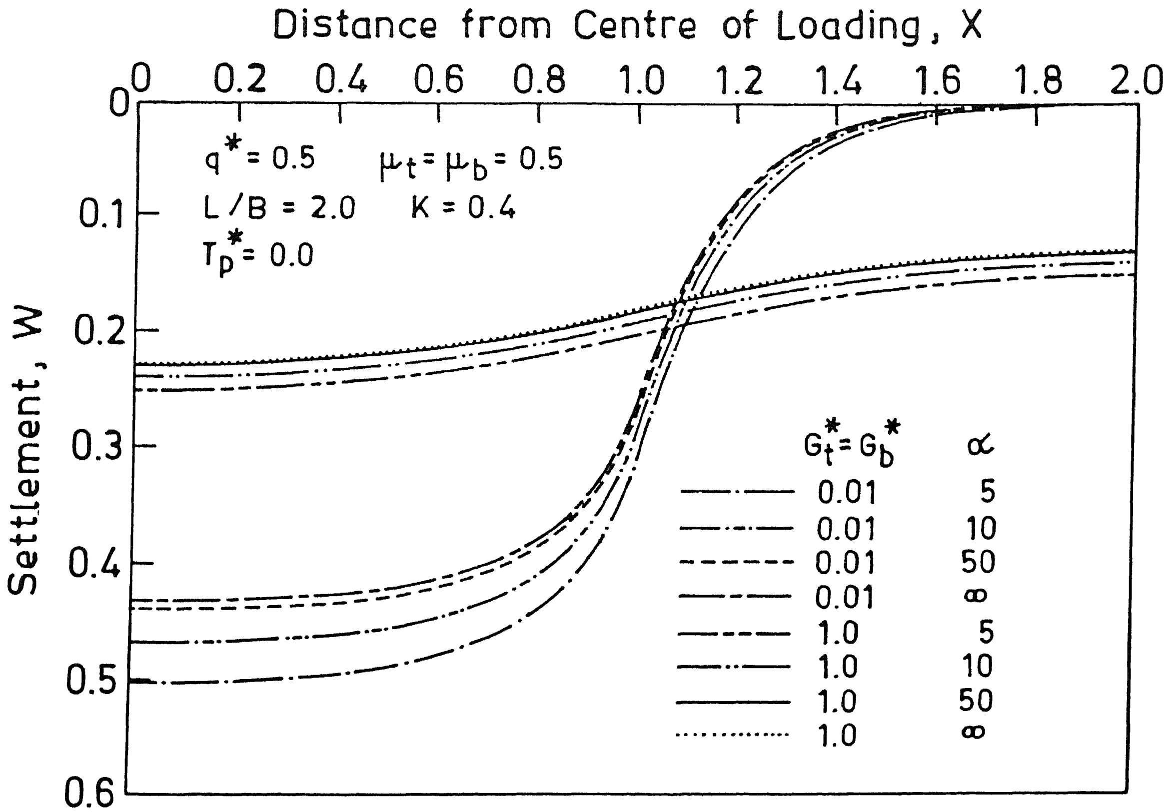


Fig. 4.19. Settlement profiles - effect of compressibility of the granular fill for its different shear parameters.

that the settlement, in general, decreases with the increase in the value of modular ratio, α, the decrease being more within the loaded region. For example, as α increases from 5 to 50, the settlement reduces by 12.75% at the centre of the loaded region and by 13.92% at the edge for $G_t^* = G_b^* = 0.01$ whereas, the corresponding settlement reductions are 8.37% and 9.85% respectively for $G_t^* = G_b^* = 1.0$. This indicates that, for the set of parameters studied, the order of decrease in the settlement for a decrease in the compressibility of the granular fill at any location within the loaded region is less for higher values of shear parameters (i.e., for higher values of thickness or shear modulus of the layers of granular fill).

Figure 4.20 shows typical settlement profiles for different interfacial friction coefficients, bringing out the effect of compressibility of the granular fill. It is observed that an increase in the value of α results in significant reduction in settlement within the loaded region. For example, as α increases from 5 to 50, the settlement reduces by 13.44% at the centre of the loaded region and by 14.68% at the edge for $\mu_t = \mu_b = 0.1$ whereas the corresponding settlement reductions are 11.43% and 13.29% respectively for $\mu_t = \mu_b = 1.0$. These results indicate that the order of settlement reduction with decreasing compressibility of the granular fill is relatively more for lower frictional coefficient at the fill-geosynthetic interface.

Figure 4.21 shows the effect of compressibility of the granular fill on the settlement profiles of geosynthetic-reinforced granular fill - soft soil system in case of prestress in the geosynthetic reinforcement. It is observed that as α increases from 5 to 50, the

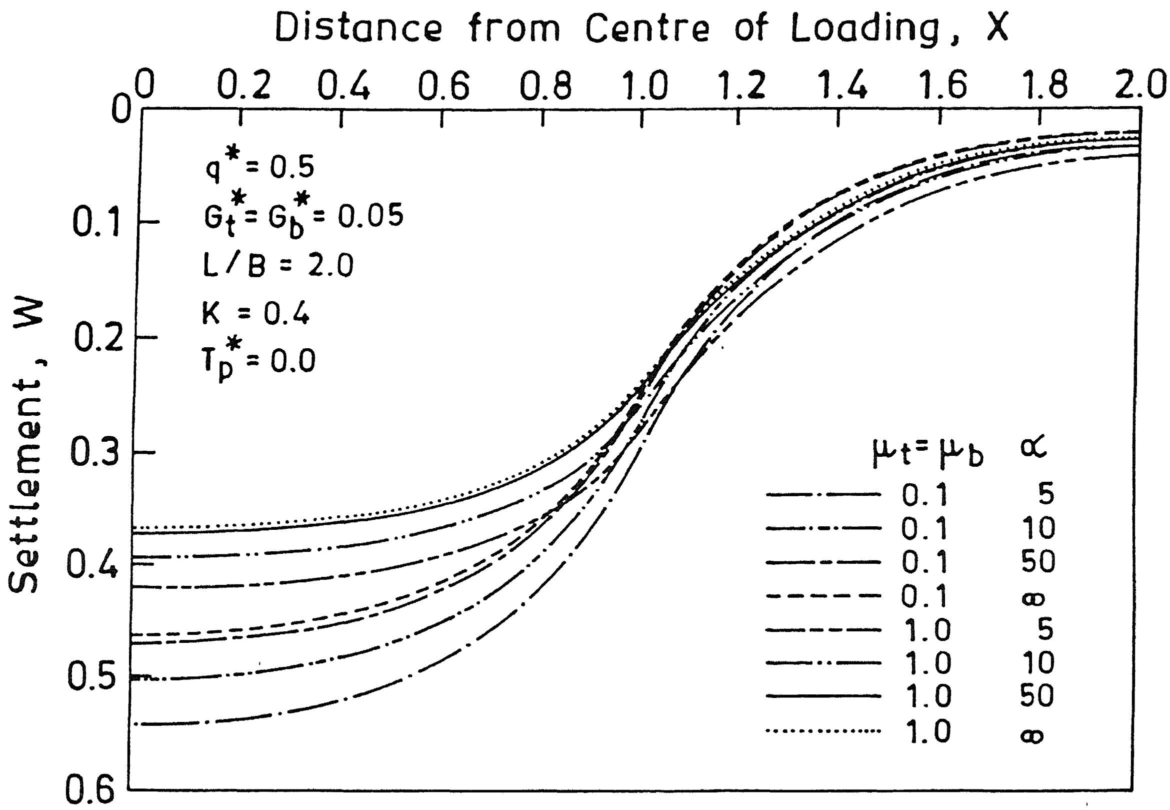


Fig. 4.20. Settlement profiles - effect of compressibility of the granular fill at different interfacial friction coefficients.

settlement reduces by 12.75% at the centre of the loaded region and by 13.92% at the edge for the case of no prestress in the reinforcement whereas, the corresponding settlement reductions are 10.79% and 13.33% respectively for the case in which geosynthetic reinforcement is prestressed. Hence if one considers the granular fill to be incompressible in settlement calculations, the error in predictions of settlement will be relatively more in case of no prestress in the geosynthetic reinforcement compared to the case in which prestress is applied in the geosynthetic reinforcement.

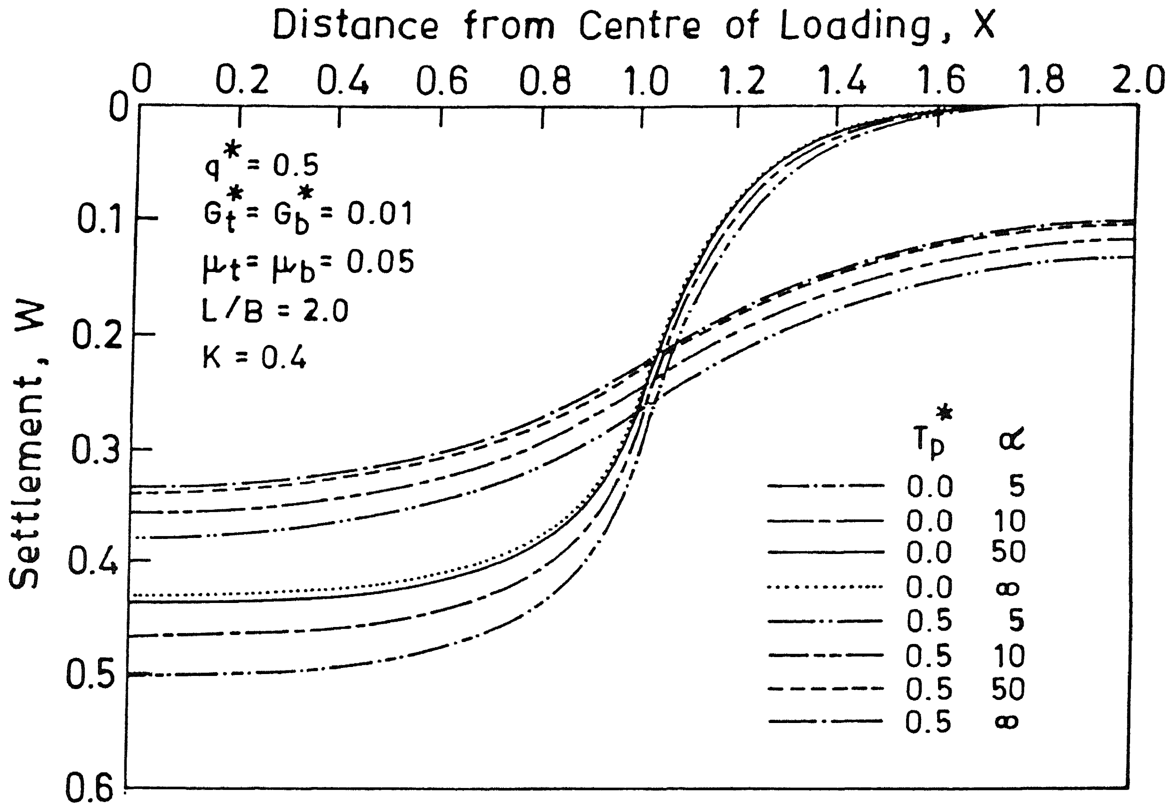


Fig. 4.21. Settlement profiles - effect of compressibility of the granular fill in case of prestress in the geosynthetic reinforcement.

4.5.4 Time-Dependent Behaviour

The results presented below reveal the effect of several parameters on the settlement response of the geosynthetic-reinforced granular fill - soft soil system at various stages of one-dimensional consolidation of the soft foundation soil.

Figure 4.22 shows the variation of the settlement at the centre of the loaded region with degree of consolidation for various load intensities. It can be observed that for the set of parameters studied, the settlement increases as the consolidation process of the soft soil proceeds. At smaller loads the settlement increases almost linearly with the increase

of degree of consolidation whereas at higher loads the rate of increase is nonlinear and decreases as the degree of consolidation increases. For example, for $q^* = 0.5$, the increase in settlement is 0.047 for the increase of U from 10% to 20% whereas, the increase is 0.039 for the increase of U from 50% to 60%.

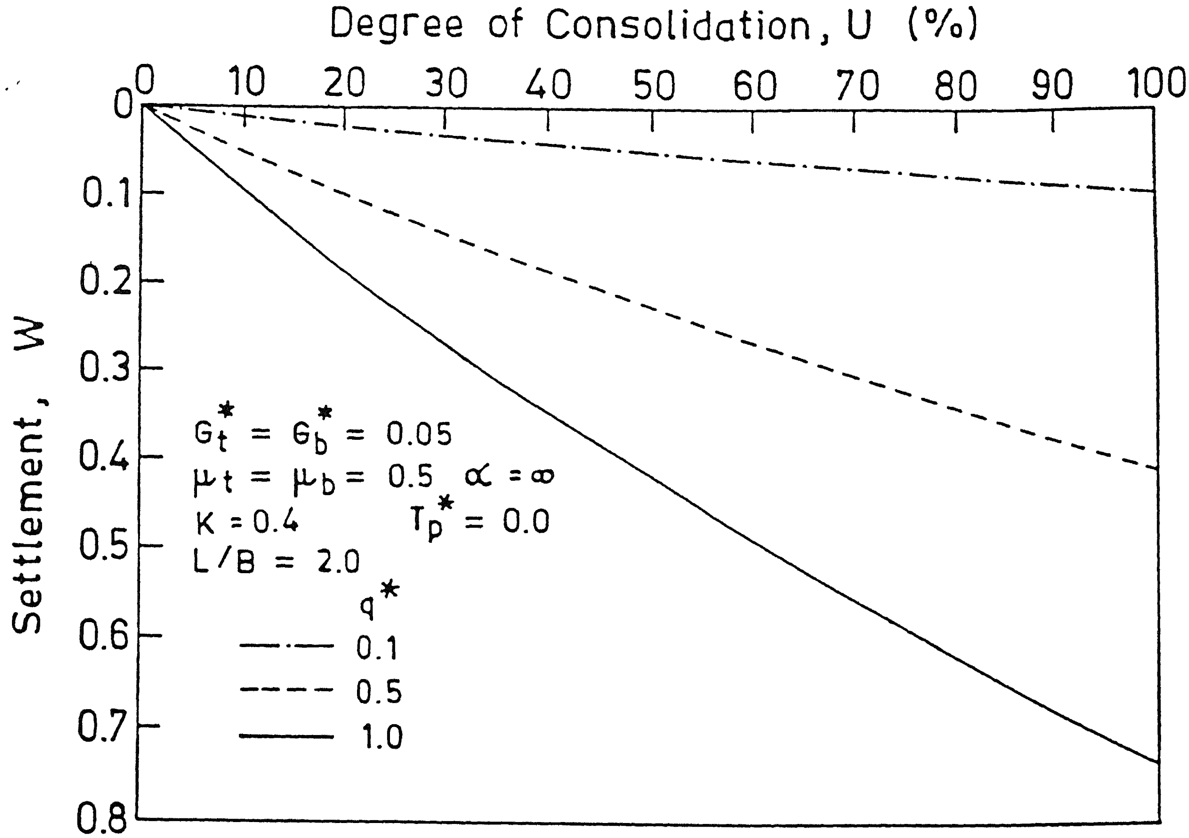


Fig. 4.22. Variation of settlement at the centre of the loaded region with degree of consolidation of the soft foundation soil for various load intensities.

Figure 4.23 shows typical settlement profiles for different nondimensional load intensities at various stages of consolidation of the soft foundation soil. As expected, the settlement under the loaded region due to consolidation of saturated soft foundation soil is relatively quite large as compared to the settlement beyond the loaded region. For

example, the difference in settlement at the centre and at the edge is 0.023 for $q^* = 0.1$ whereas the difference is 0.162 for $q^* = 1.0$ for 50% consolidation. The corresponding differences are 0.027 and 0.235 respectively for 90% consolidation.

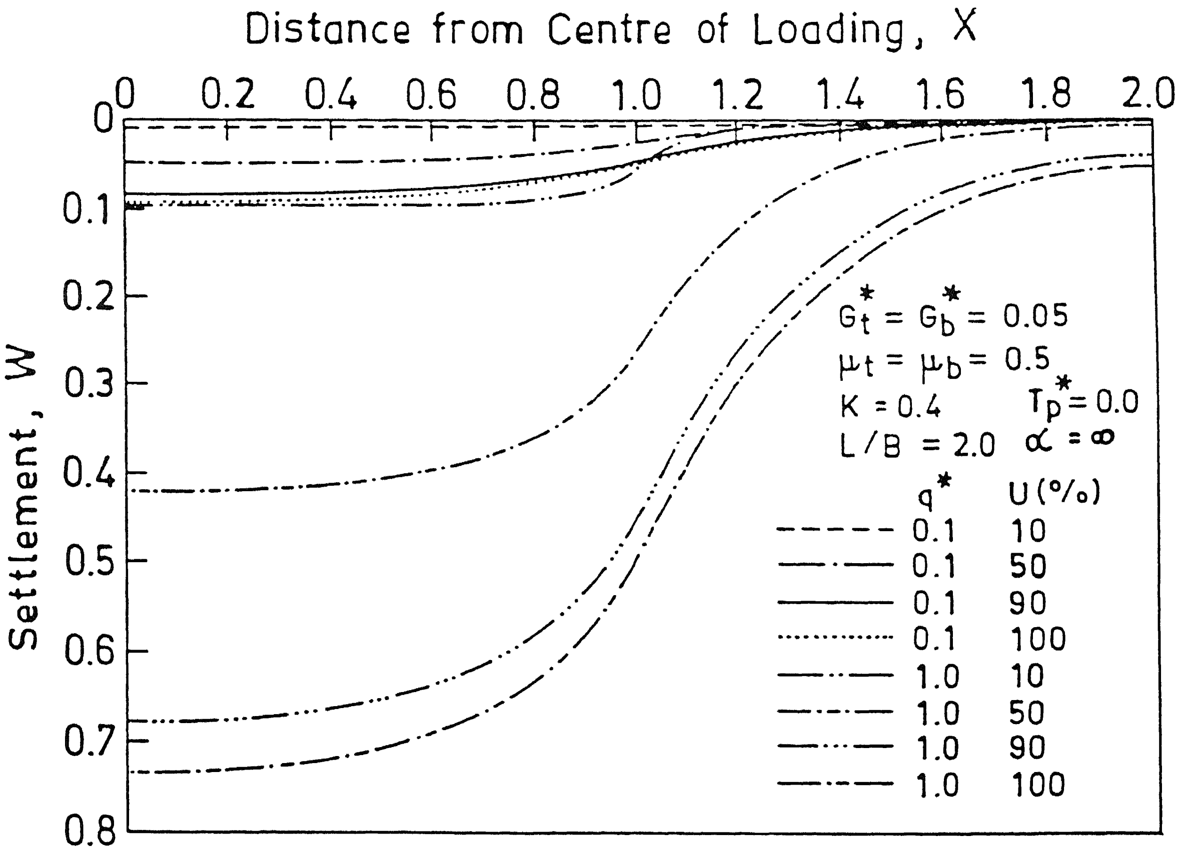


Fig. 4.23. Settlement profiles for different load intensities at various stages of consolidation of the soft foundation soil.

Figure 4.24 shows typical settlement profiles for different nondimensional shear parameters, at various stages of consolidation of the soft foundation soil. It is observed that the effect of consolidation is more predominant under the loaded region for granular fill with low shear parameter as compared with the granular fill with high shear parameter where the increase in settlement is observed throughout. For example, as the degree of

consolidation increases from 50 to 90%, the settlement at the centre increases by 68.38% for $G_t^* = G_b^* = 0.01$ and by 44.59% for $G_t^* = G_b^* = 1.0$.

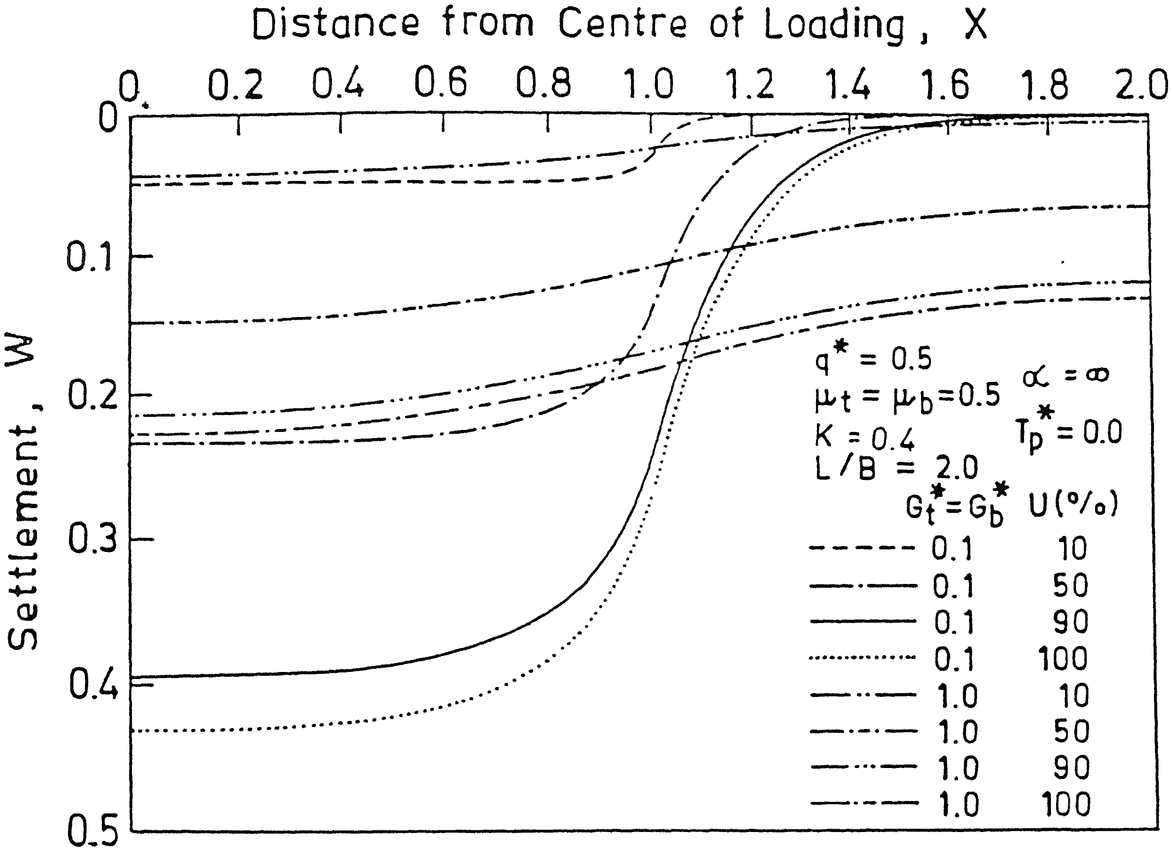


Fig. 4.24. Settlement profiles for different shear parameters of granular fill at various stages of consolidation of the soft foundation soil.

Figure 4.25 shows typical settlement profiles for different interfacial friction coefficients at various stages of consolidation of the soft foundation soil. It is noticed that during initial stages of consolidation, the settlement profile is independent of interface friction coefficient. For higher degree of consolidation the settlement under the loaded region is less for higher interface friction coefficient whereas beyond the loaded region the trend is reversed. For example, as the degree of consolidation increases from 50

to 90%, the settlement at the centre increases by 72.84 % for $\mu_t = \mu_b = 0.1$ and by 57.14% for $\mu_t = \mu_b = 1.0$.

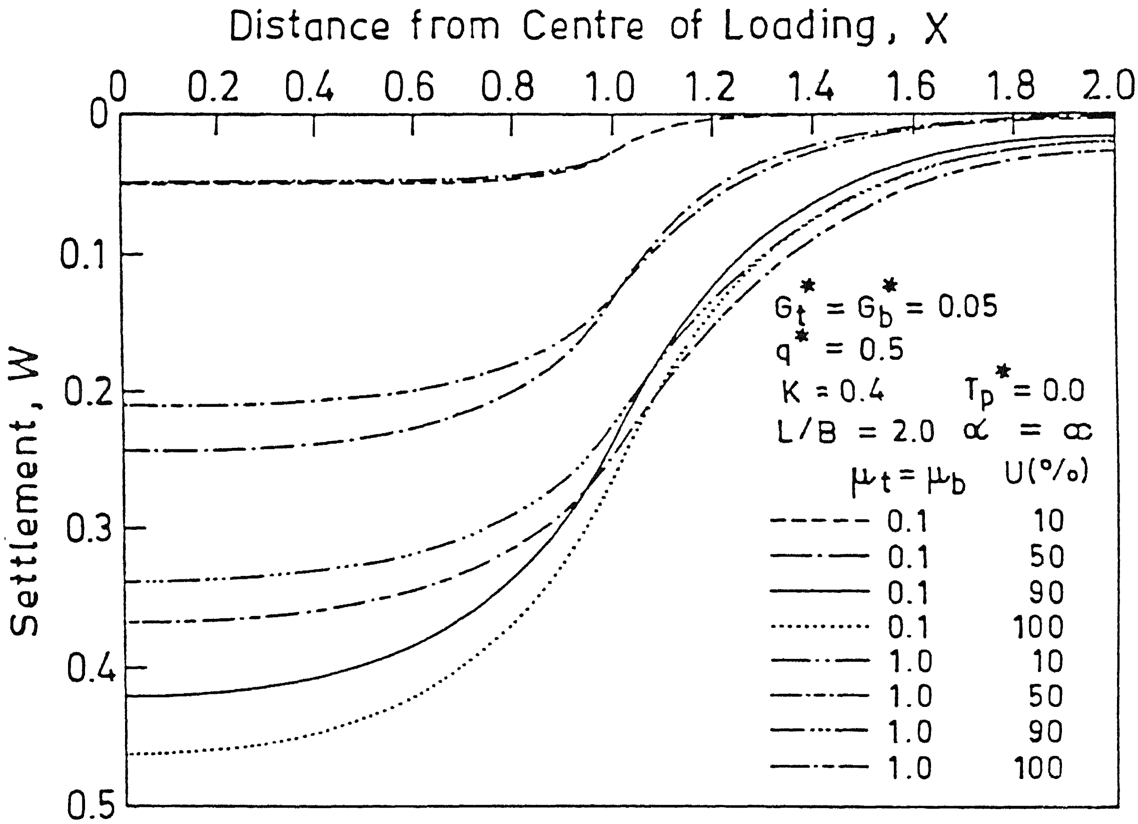


Fig. 4.25. Settlement profiles for different interfacial friction coefficients at various stages of consolidation of the soft foundation soil.

Figure 4.26. shows typical settlement profiles for different widths of reinforced zone at various stages of consolidation of soft foundation soil. It is observed that for lower degrees of consolidation, the settlement profiles are almost identical for both values of width of reinforced zone. For higher degrees of consolidation the settlement at any location is relatively less for higher width of reinforced zone. For example, the settlement

at the centre of the loaded region decreases by 1.84 % for U = 90% and by 2.16% for U = 100% as the width of reinforced zone is increased from 1.2 to 2.0.

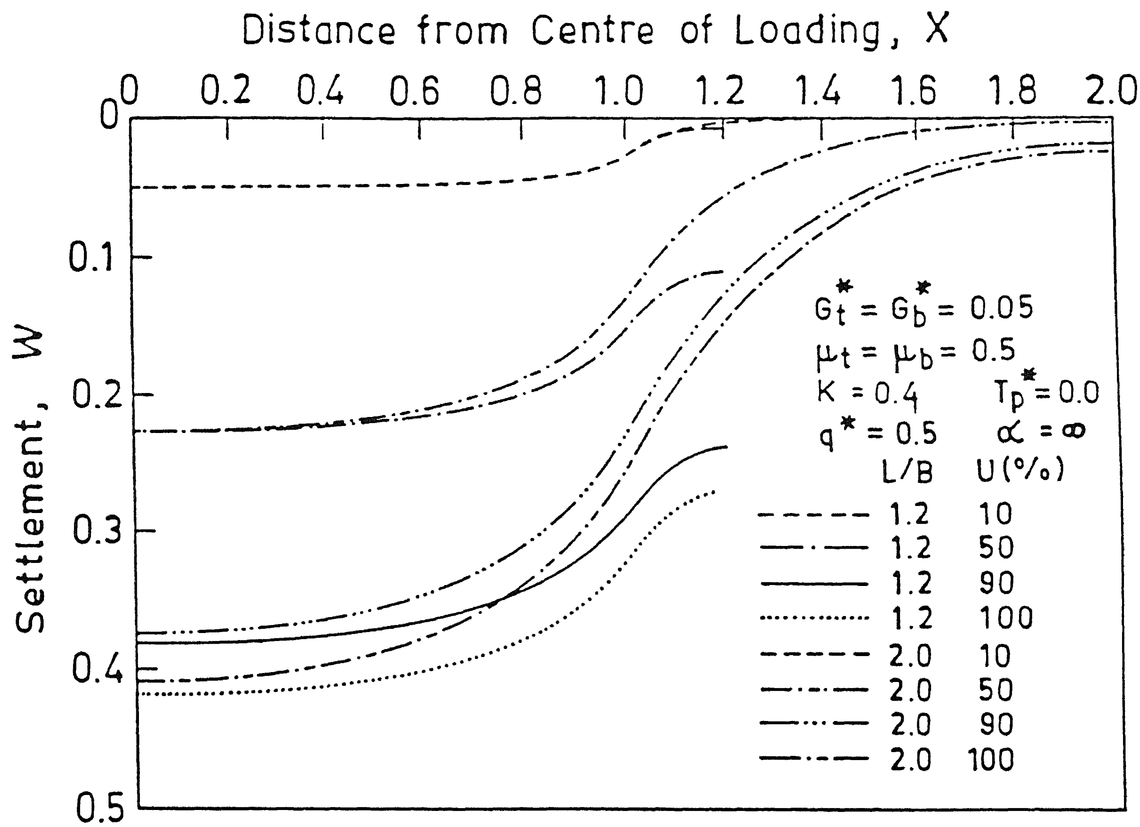


Fig. 4.26. Settlement profiles for different widths of reinforced zone at various stages of consolidation of the soft foundation soil.

Figure 4.27 shows the effect of lateral stress ratio on the settlement profiles at various stages of consolidation of the soft foundation soil. It is noticed that in the beginning of the consolidation no beneficial effect of higher horizontal stresses in the granular fill is observed. For higher degree of consolidation of the soft foundation soil, the settlement at any location is relatively less if granular fill is having higher lateral stress ratio. For example, the settlement at the centre of the loaded region decreases by 0.00%, 1.78%,

3.21% and 3.68% for 10%, 50%, 90% and !00% consolidation as K is increased from 0.4 to 0.7.

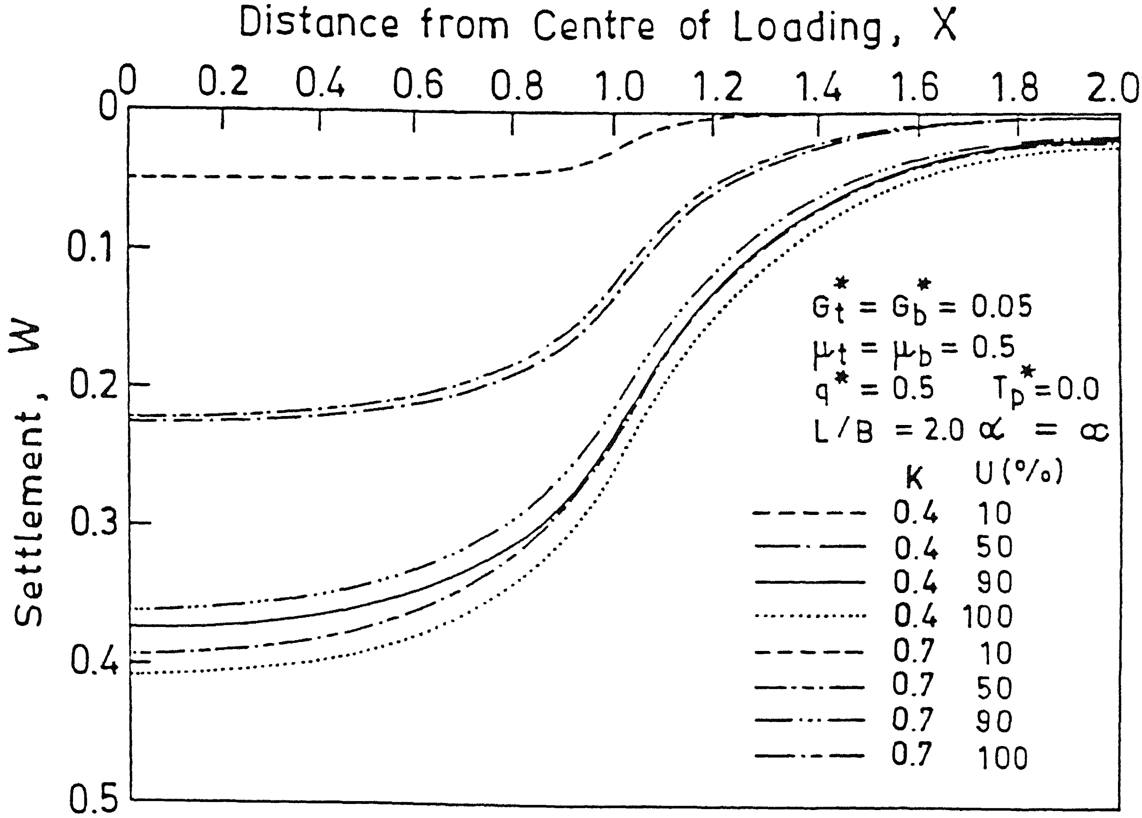


Fig. 4.27. Settlement profiles for different lateral stress ratios in the granular fill at various stages of consolidation of the soft foundation soil.

Figure 4.28 shows the effect of prestress in the geosynthetic reinforcement on the settlement profiles at various stages of consolidation of the soft foundation soil. It is noticed that the settlement at any location within the loaded region at any stage of consolidation is less for the case of prestress in the geosynthetic reinforcement than the case in which no prestressing is used whereas, beyond the loaded region the trend is reversed. For example, for 50% consolidation, the settlement at the centre of the loaded

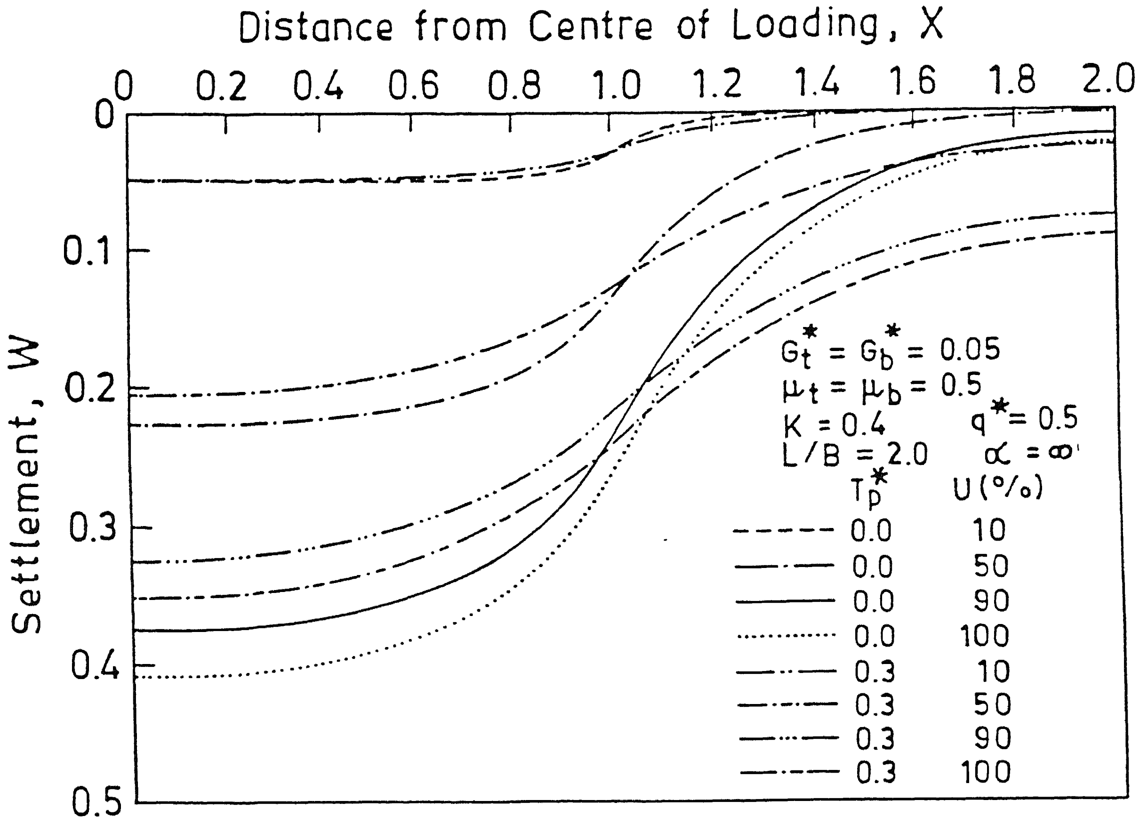


Fig. 4.28. Settlement profiles for different prestress values in the geosynthetic reinforcement at various stages of consolidation of the soft soil.

consolidation. The corresponding reductions in settlement at the edge of the loaded region are 4.58 and 6.14% respectively.

Figure 4.29 shows typical settlement profiles for different modular ratios at various stages of consolidation of the soft foundation soil. It is observed that at any stage of consolidation of soft foundation soil, the settlement at any location is more for lower value of α than that for higher value of α.

The variation of the mobilised tensile force in the geosynthetic reinforcement along the length of reinforcement at various stages of consolidation of soft foundation soil has

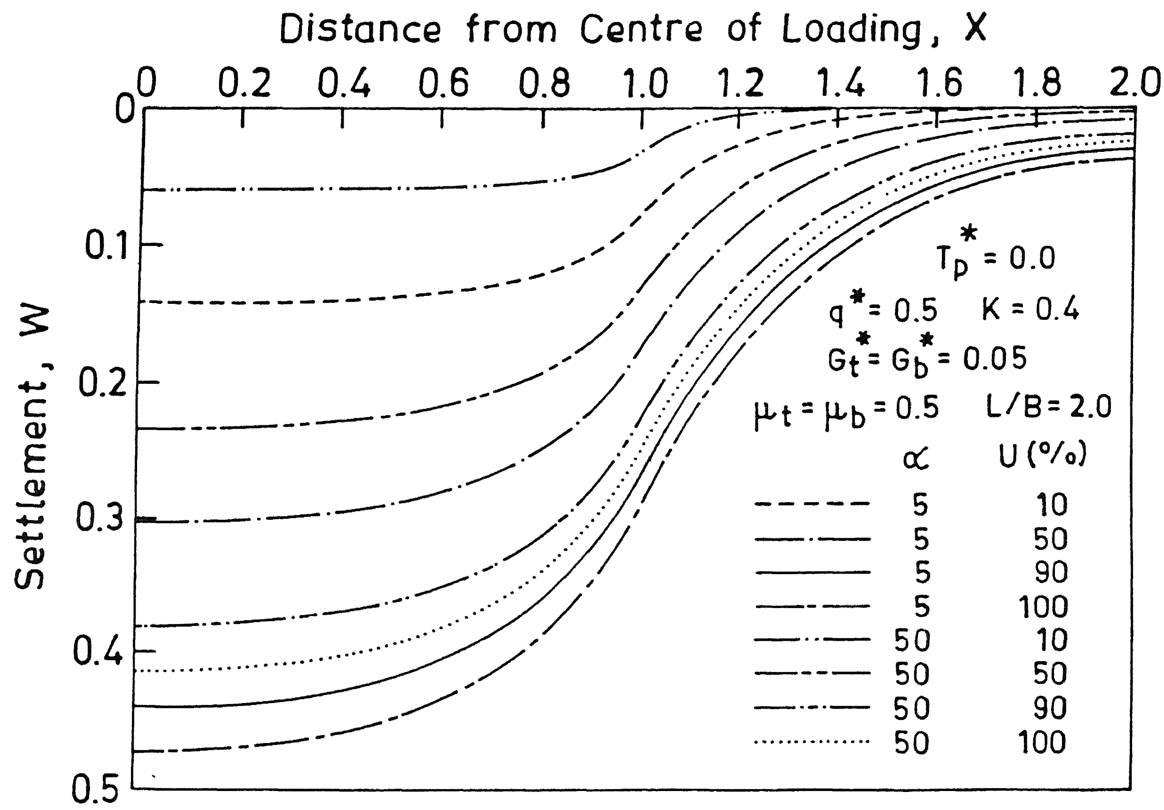


Fig. 4.29. Settlement profiles for different modular ratios at various stages of consolidation of the soft foundation soil.

been shown in Fig. 4.30. It is observed that the mobilised tension at the centre of the loaded region has the highest value in the beginning of the consolidation process. This is because of the rigid-plastic friction model considered in the present study which mobilises full frictional resistance for any finite settlement greater than zero. Further, it can be noticed that there is a slight decrease of mobilised tensile force near the centre and increase beyond this as the degree of consolidation increases.

To understand this behaviour in a better way, the variations of stresses at the top and bottom of the geosynthetic layer have been plotted both at the centre and at the edge in Fig. 4.31. The stress below the geosynthetic layer decreases at the centre of the

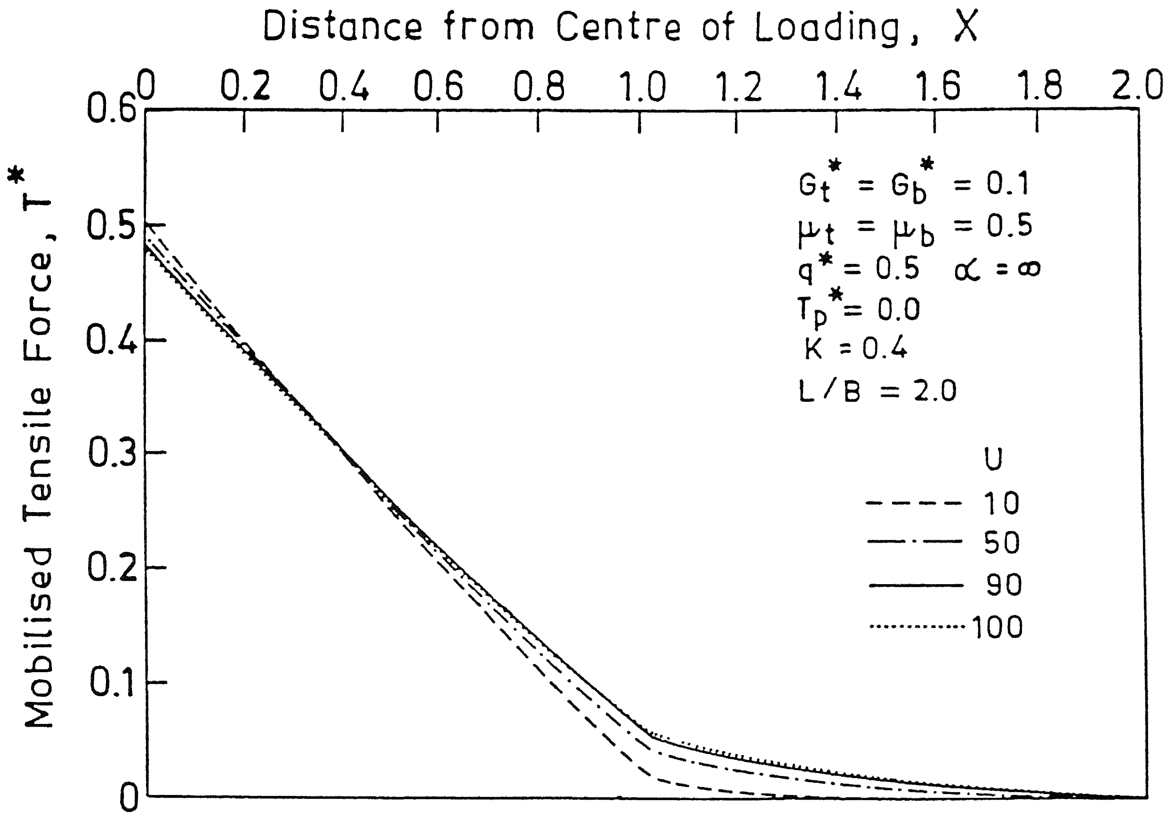


Fig. 4.30. Mobilised tensile force distribution profiles at various stages of consolidation of the soft foundation soil.

loaded region whereas it increases at the edge as the settlement increases with increasing degree of consolidation. Further, it can be noted that the stress at the top of the geosynthetic layer remains almost constant with the increase of degree of consolidation both at the centre and at the edge of the loaded region.

4.6. CONCLUSIONS

The proposed foundation model has been employed successfully for the study of settlement behaviour of geosynthetic-reinforced granular fill - soft soil system in plane

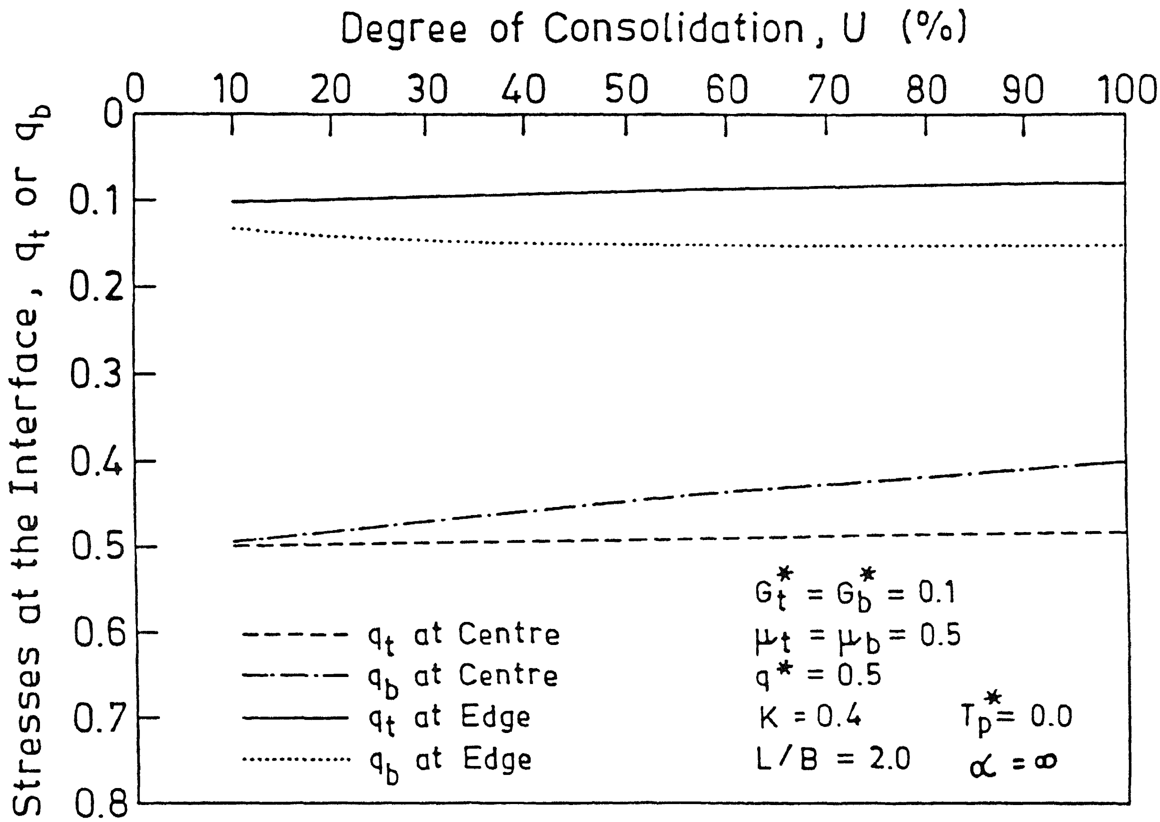


Fig. 4.31. Variation of normal stresses at the top and the bottom faces of the geosynthetic reinforcement at various stages of consolidation.

strain conditions. The detailed parametric study carried out has revealed several key features of the settlement behaviour which has been discussed in the previous section. The consideration of vertical shear stress transfer at the fill-geosynthetic interface in addition to the horizontal shear stress transfer in the present model has made it applicable for solving the problems involving relatively large deformations as well where existing mechanical models for geosynthetic-reinforced soil are not applicable for the range of nondimensional parameters studied. Comparisons of the settlement predictions by the present foundation model with those computed using the existing mechanical foundation models have showed similar trend of results for similar model parameters.

The effects of lateral stress ratio of the granular fill, and the prestressing of the geosynthetic reinforcement have been quantified. Prestressing the geosynthetic reinforcement has been found to be very effective in reducing both the total and differential settlements of the loaded region. It has been found that the compressibility of the granular fill has an appreciable influence on the settlement response of the geosynthetic-reinforced granular fill soft soil system as long as the stiffness of the granular fill is less than approximately 50 times that of the soft soil.

At smaller loads the settlement increases almost linearly with the increase of degree of consolidation of the soft foundation soil whereas, at higher loads the rate of increase is nonlinear and decreases as degree of consolidation increases. The effect of consolidation of the soft foundation soil is more predominant under the loaded region for granular fill with low shear parameter. During initial stages of consolidation the settlement profile is almost independent of interface friction coefficients, width of reinforced zone, prestress in the geosynthetic reinforcement, lateral stress ratio etc.

CHAPTER 5

SETTLEMENT ANALYSIS - AXI-SYMMETRIC CASE

5.1 INTRODUCTION

In practical applications, the geosynthetic-reinforced granular fill - soft soil system are often used as foundations for tank filled with liquid, parking lots, warehouses and columns, circular in plan. The analyses of such foundations are relatively more complex and challenging and are attempted by considering these as axi-symmetric problems. The equations governing the response of the proposed model at any particular instant of time ($t>0$) are derived by considering the equilibrium of forces on different elements of the reinforced soil system at that instant. The parametric studies are carried out to observe the effects of various parameters on the load-settlement response of the reinforced soil system. Wherever it is possible, the present results are compared with those computed using the existing mechanical models. The numerical procedure adopted is similar to the one adopted for the analysis of plane strain conditions, and the results are presented in the nondimensional form.

5.2 RESPONSE FUNCTION OF THE MODEL

Figure 5.1(a) shows a geosynthetic-reinforced granular fill - soft soil system subjected to axi-symmetric loading. The behaviour of such a system is idealised by the proposed

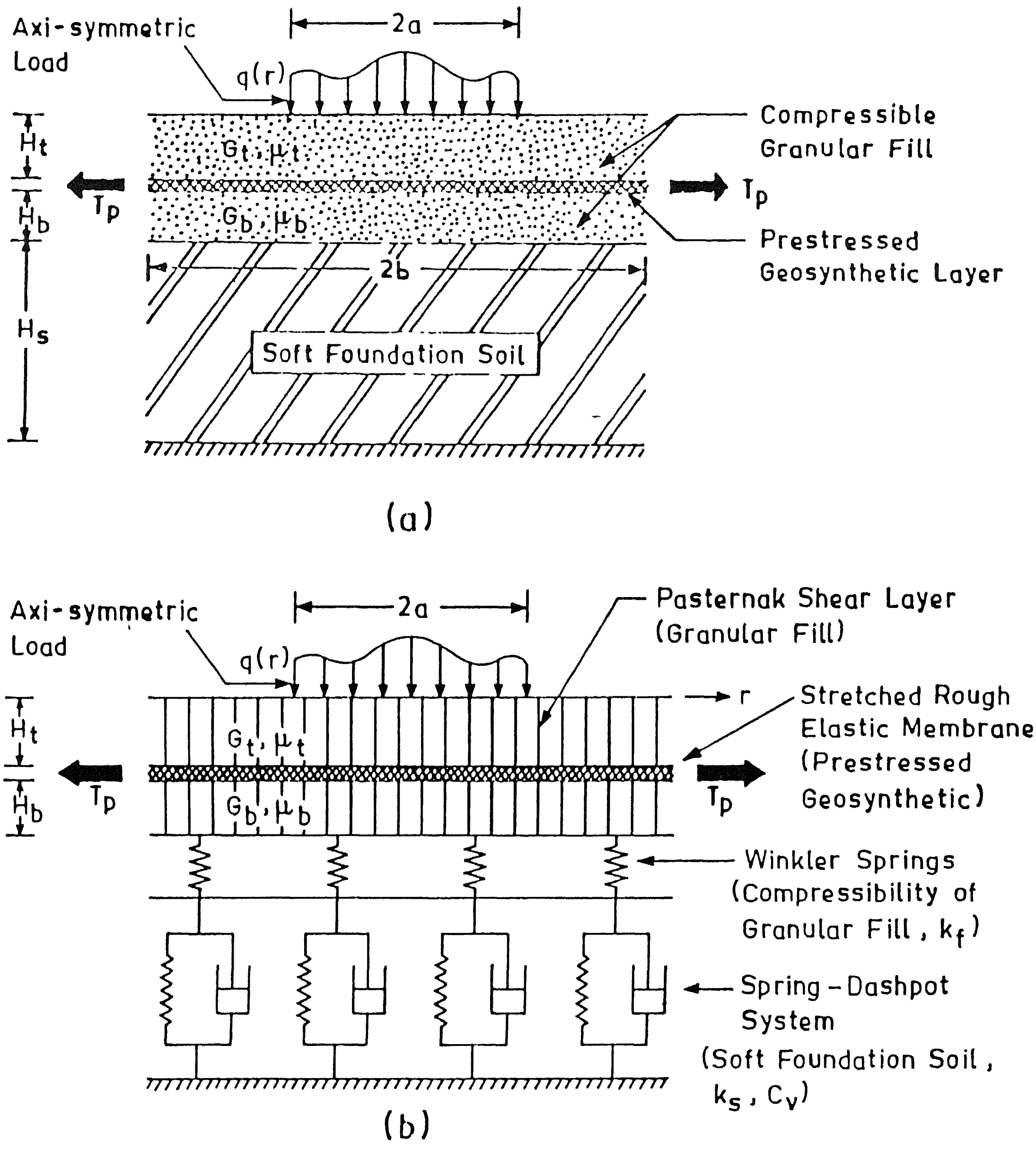


Fig. 5.1. Definition sketch: (a) geosynthetic-reinforced granular fill - soft soil system subjected to axi-symmetric loading; (b) proposed foundation model.

foundation model as shown in Fig. 5.1(b) along with the system of co-ordinates considered in the derivation of governing equations of the response of the model.

The vertical force equilibrium equations of the upper and lower shear layer elements at time t>0 (Fig. 5.2 (b) & (c)), can be written as:

$$q = q_t - G_t H_t \left(\frac{\partial^2 w}{\partial r^2} + \frac{1}{r}\frac{\partial w}{\partial r}\right) \tag{5.1}$$

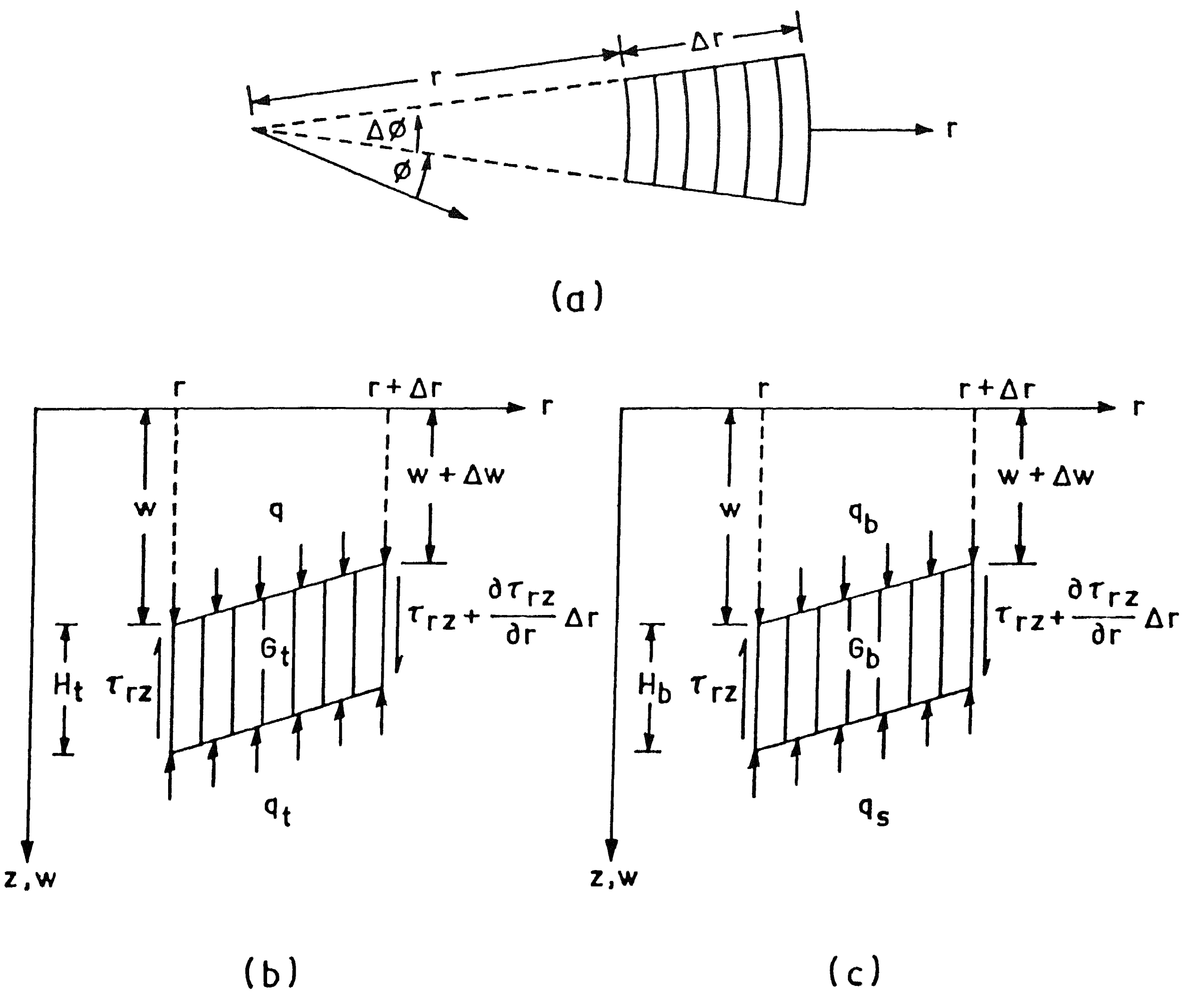


Fig. 5.2. Definition sketch: (a) plan view of shear elements; (b) forces on the upper shear layer element; (c) forces on the lower shear layer element.

and

$$q_b = q_s - G_b H_b \left(\frac{\partial^2 w}{\partial r^2} + \frac{1}{r}\frac{\partial w}{\partial r}\right) \tag{5.2}$$

where, r is the distance measured from the centre of the axi-symmetrically loaded region along radial direction.

The expression for q_s can be given as:

$$q_s = \frac{\alpha\, k_s\, w}{1 + \alpha\, U} \tag{5.3}$$

The horizontal force equilibrium equation of the stretched rough elastic membrane element at time t>0 (Fig. 5.3(b)), can be written as:

$$(T_p + T + \Delta T)(r + \Delta r)\Delta\phi \cos(\theta + \Delta\theta) - (T_p + T) r \Delta\phi \cos\theta + K(q_t - q_b)\Delta r \tan\theta \left(r + \frac{\Delta r}{2}\right)\Delta\phi$$

$$- (\mu_t q_t + \mu_b\, q_b)\Delta r \left(r + \frac{\Delta r}{2}\right)\Delta\phi = 0 \tag{5.4}$$

As $\Delta r \to 0$, eqn (5.4) reduces to:

$$r\cos\theta \frac{\partial T}{\partial r} + (T_p + T)\cos\theta - (T_p + T) r \sin\theta \frac{\partial\theta}{\partial r} = -r\{(\mu_t q_t + \mu_b\, q_b) +$$

$$K(q_t - q_b)\tan\theta\} = 0 \tag{5.5}$$

The vertical force equilibrium equation for the stretched rough elastic membrane element at time t>0 (Fig. 5.3(b)) can be written as:

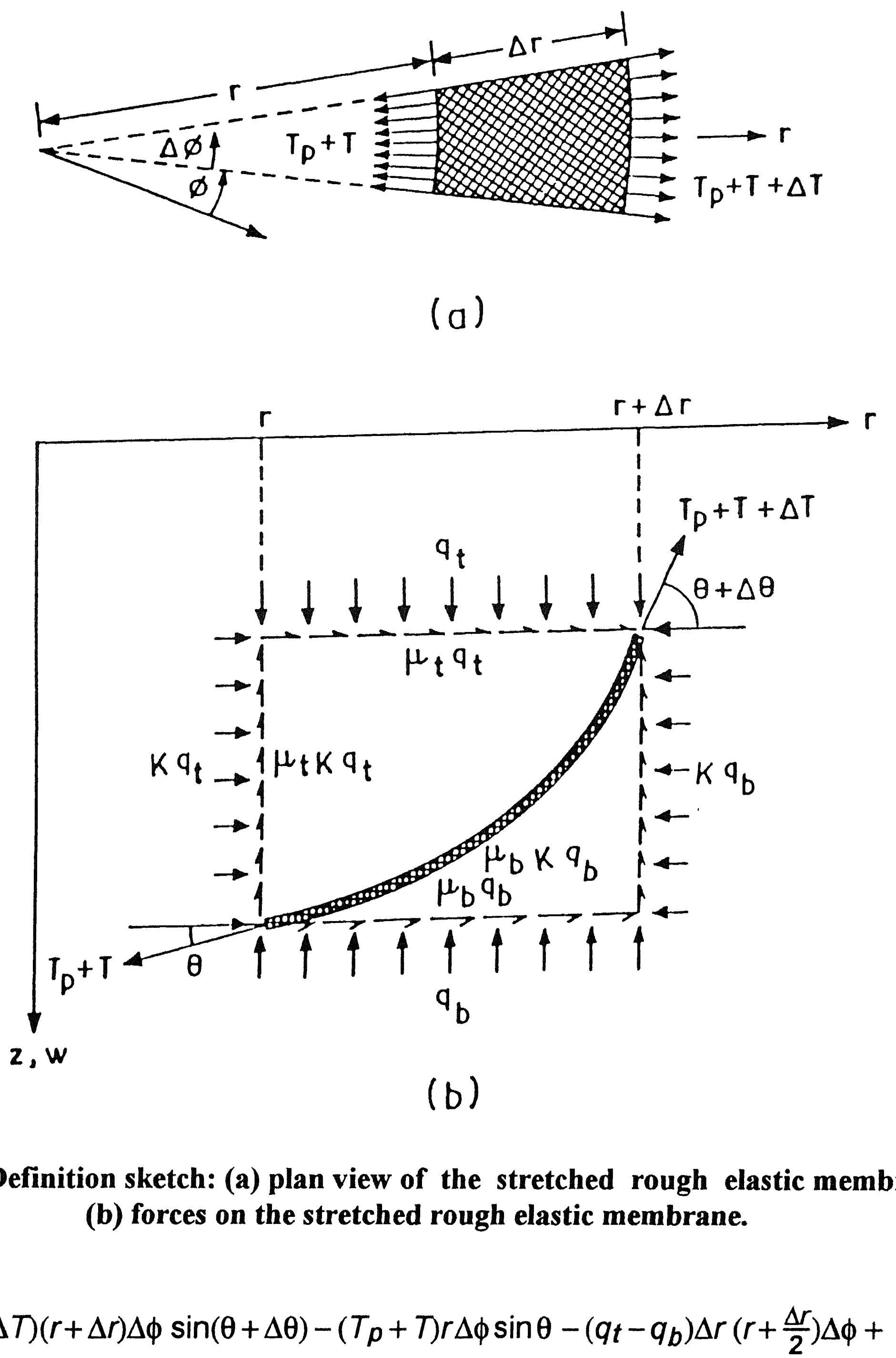


Fig. 5.3 Definition sketch: (a) plan view of the stretched rough elastic membrane; (b) forces on the stretched rough elastic membrane.

$$(T_p+T+\Delta T)(r+\Delta r)\Delta\phi\,\sin(\theta+\Delta\theta)-(T_p+T)r\Delta\phi\sin\theta-(q_t-q_b)\Delta r\,(r+\tfrac{\Delta r}{2})\Delta\phi+$$

$$K(\mu_t q_t+\mu_b q_b)\Delta r\tan\theta\,(r+\tfrac{\Delta r}{2})\Delta\phi=0 \tag{5.6}$$

As $\Delta r\to 0$, eqn (5.6) reduces to:

$$r\sin\theta\frac{\partial T}{\partial r}+(T_p+T)\sin\theta+(T_p+T)r\cos\theta\frac{\partial\theta}{\partial r}=r\{(q_t-q_b)-K(\mu_t q_t+\mu_b q_b)\tan\theta\} \quad (5.7)$$

Using eqns (5.5) and (5.7), one can write:

$$q_t=q_b+\frac{(T_p+T)\sec\theta}{1+K\tan^2\theta}\frac{d\theta}{dr}-\frac{(\mu_t q_t+\mu_b q_b)(1-K)\tan\theta}{1+K\tan^2\theta} \quad (5.8)$$

Substituting for $d\theta/dr$ in terms of the vertical surface displacement, w(r, t) into eqn (5.8), one can write:

$$q_t=\overline{X}_1 q_b-\overline{X}_2(T_p+T)\cos\theta\frac{\partial^2 w}{\partial r^2} \quad (5.9)$$

where,

$$\overline{X}_1=\frac{1+K\tan^2\theta-(1-K)\mu_b\tan\theta}{1+K\tan^2\theta+(1-K)\mu_t\tan\theta} \quad (5.10a)$$

and

$$\overline{X}_2=\frac{1}{1+K\tan^2\theta+(1-K)\mu_t\tan\theta} \quad (5.10b)$$

Combining eqns (5.1) - (5.3) and (5.9), one can write:

$$q=\overline{X}_1\frac{\alpha k_s w}{1+\alpha U}-\overline{X}_2(T_p+T)\cos\theta\frac{\partial^2 w}{\partial r^2}-(G_t H_t+\overline{X}_1 G_b H_b)\left(\frac{\partial^2 w}{\partial r^2}+\frac{1}{r}\frac{\partial w}{\partial r}\right) \quad (5.11)$$

Combining eqn (5.1) - (5.3), (5.5) and (5.7), one can write:

$$\frac{\partial T}{\partial r}+\frac{T_p+T}{r}=-\overline{X}_3\{q+G_t H_t(\frac{\partial^2 w}{\partial r^2}+\frac{1}{r}\frac{\partial w}{\partial r})\}-\overline{X}_4\{\frac{\alpha k_s w}{1+\alpha U}-G_b H_b(\frac{\partial^2 w}{\partial r^2}+\frac{1}{r}\frac{\partial w}{\partial r})\} \quad (5.12)$$

where,

$$\overline{X}_3 = \mu_t \cos\theta(1 + K\tan^2\theta) - (1 - K)\sin\theta \qquad (5.13a)$$

and

$$\overline{X}_4 = \mu_b \cos\theta(1 + K\tan^2\theta) + (1 - K)\sin\theta \qquad (5.13b)$$

Equations (5.11) and (5.12) govern the response of the proposed foundation model.

5.3. METHOD OF SOLUTION

To observe the settlement response of the proposed model, eqns (5.11) and (5.12) are stated in their nondimensional forms as:

$$q^* = \overline{X}_1 \frac{\alpha W}{1+\alpha U} - \overline{X}_2 (T_p^* + T^*)\cos\theta \frac{\partial^2 W}{\partial R^2} - (G_t^* + \overline{X}_1 G_b^*)\left(\frac{\partial^2 W}{\partial R^2} + \frac{1}{R}\frac{\partial W}{\partial R}\right) \qquad (5.14)$$

and

$$\frac{\partial T^*}{\partial R} + \frac{T_p^* + T^*}{R} + \overline{X}_3\left\{q^* + G_t^*\left(\frac{\partial^2 W}{\partial R^2} + \frac{1}{R}\frac{\partial W}{\partial R}\right)\right\} + \overline{X}_4\left\{\frac{\alpha W}{1+\alpha U} - G_b^*\left(\frac{\partial^2 W}{\partial R^2} + \frac{1}{R}\frac{\partial W}{\partial R}\right)\right\} = 0$$

(5.15)

where $R = r/a$, $W = w/a$, $G_t^* = G_t H_t / k_s a^2$, $G_b^* = G_b H_b / k_s a^2$, $q^* = q/k_s a$, $T_p^* = T_p/k_s a^2$, and $T^* = T/k_s a^2$, in which a is the radius of the loaded region.

Writing eqns (5.14) and (5.15) in finite difference form within the specified time-space domain, for an interior node, (i, j), where i, j are indices for space and time respectively, one gets:

$$q_{i,j}^{*} = \overline{X}_{1i,j} \frac{\alpha W_{i,j}}{1+\alpha U_j} - \overline{X}_{2i,j}(T_p^{*} + T_{i,j}^{*})\cos\theta_{i,j} \frac{d^2W}{dR^2}\Big|_{i,j} - (G_t^{*} + \overline{X}_{1i,j}\, G_b^{*})(\frac{d^2W}{dR^2}\Big|_{i,j}$$

$$+\frac{1}{R_i}\frac{dW}{dR}\Big|_{i,j})\} \tag{5.16}$$

and

$$T_{i,j}^{*} = \frac{1}{1-\frac{\Delta R}{R_i}}[T_{i+1,\,j}^{*} + T_p^{*}\frac{\Delta R}{R_i} + (\Delta R/4)[(\overline{X}_{3i,j} + \overline{X}_{3i+1,j})\{(q_i^{*} + q_{i+1}^{*}) + G_t^{*}((\frac{d^2W}{dR^2} +$$

$$\frac{1}{R_i}\frac{dW}{dR})\Big|_{i,j} + (\frac{d^2W}{dR^2} + \frac{1}{R}\frac{dW}{dR})\Big|_{i+1,\,j})\} + (\overline{X}_{4i,j} + \overline{X}_{4i+1,j}))\{\frac{\alpha}{1+\alpha U_j}(W_{i,j} + W_{i+1;\,j}) -$$

$$-G_b^{*}((\frac{d^2W}{dR^2} + \frac{1}{R}\frac{dW}{dR})\Big|_{i,j} + (\frac{d^2W}{dR^2} + \frac{1}{R}\frac{dW}{dR})\Big|_{i+1,\,j})\}]] \tag{5.17}$$

In equation (5.17), the derivatives $\frac{d^2W}{dR^2}$ and $\frac{dW}{dR}$ are expressed by central finite difference scheme while $\frac{dT^*}{dR}$ has been expressed by forward finite difference scheme. Hence, in order to minimise the numerical error, average values of $\overline{X}_3$, $\overline{X}_4$, q^*, W and $(\frac{d^2W}{dR^2} + \frac{1}{R}\frac{dW}{dR})$, for each element, are taken in eqn (5.17).

Loading and boundary conditions:

The solutions are obtained for a uniform nondimensional load intensity, q_0^* acting over a circular region of diameter 2a. Since there is a symmetry about the centre of the loaded region, the slope of the settlement profile at the centre of the loaded region is taken as zero. At the edge of the reinforced zone (i.e. at R = b/a, b being the radius of the reinforced zone) the slope is considered as zero as observed in most of the practical cases where the membrane is free or fixed. The mobilised tensile force at the edge of the

reinforcement is considered as zero (i. e. $T^* = 0$ for R= b/a) which implies that the frictional resistance mobilised over the length of the membrane is sufficient to balance tensile force in it. In field, the non-zero values of mobilised tensile force may occur at the edge of the membrane if frictional resistance is not sufficient to balance the tensile force in it. The influence of non-zero values of T^* at R = b/a can also be studied by specifying the particular value of T^* as the boundary condition in the suggested model.

Based on the above formulation, results are obtained using HP 9000/850 computer system. Due to symmetry of the problem analysed, only half region of the problem ($R \geq 0$) is considered. The solutions are obtained with a convergence criterion as (Chapra and Canale, 1989):

$$\left|\frac{W_i^k - W_i^{k-1}}{W_i^k}\right| X\, 100\% < \varepsilon_S \qquad (18)$$

for all i, where k and k -1 are the present and previous iterations, and ε_s is the specified tolerance which is considered to be 0.0001 in the present study. The ranges of nodimensional parameters studied are same as those described in Table 4.1.

5.4 COMPARISON WITH EXISTING MECHANICAL MODEL

The settlements obtained by the present mechanical foundation model are compared with those obtained using the mechanical foundation model suggested by Ghosh (1991). Throughout the comparison, $K = K_0 = 0.4$, $T_p^* = 0$, $\alpha = \infty$, and U = 100% are considered in the present model to make the comparison under similar situations.

Figure 5.4 shows typical settlement profiles of geosynthetic-reinforced granular fill - soft soil system for various nondimensional load intensities. The settlement profiles as obtained by using the model suggested by Ghosh (1991), which considers only the

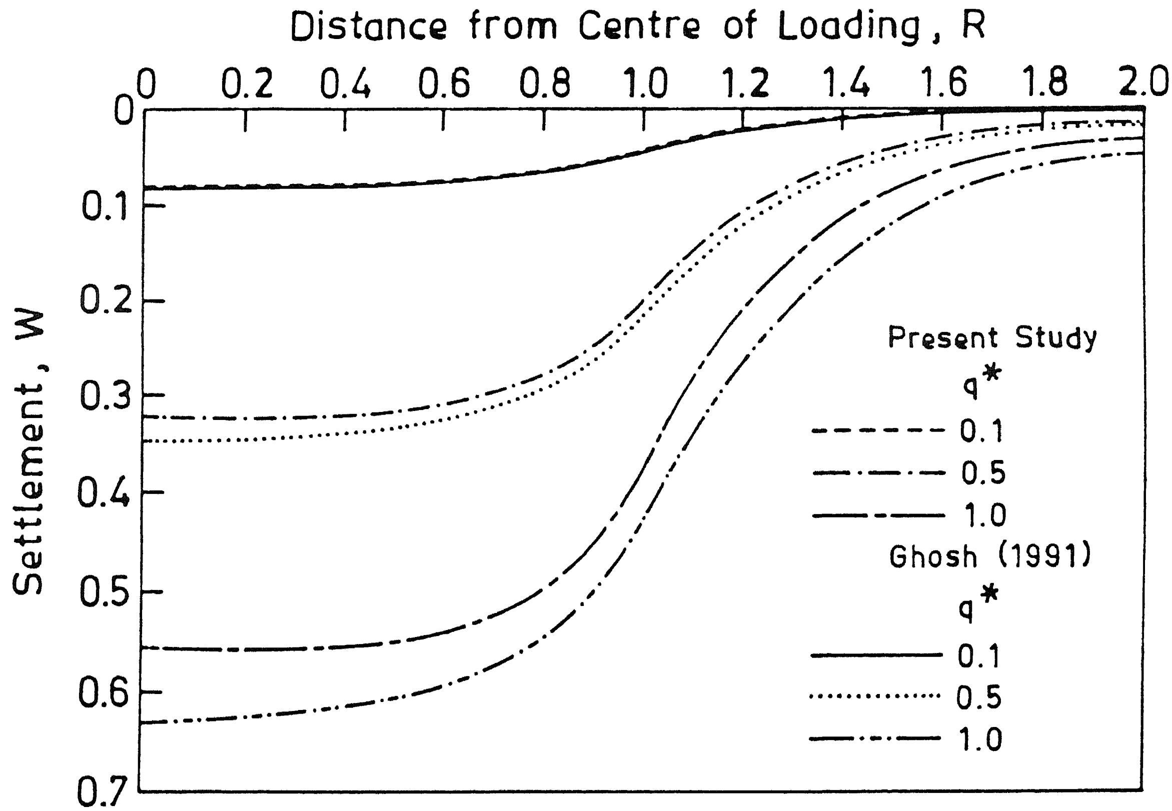


Fig. 5.4. Settlement profiles - comparison of foundation models.

horizontal shear stress transfer at the fill-geosynthetic interface, are plotted for the sake of comparison. It is noticed that for very small load intensity ($q^* = 0.1$), the settlement profile obtained by the present model almost coincides with the settlement profile obtained by Ghosh model. However, the settlement predictions by both the models differ significantly for higher load intensities. At any location, the present model predicts less settlement than

that predicted by Ghosh model. For example, for $q^* = 0.5$ and 1.0, the settlement predictions by the present model are less by 7.47% and 12.04% at the centre of the loaded region whereas at the edge of the loaded region the corresponding values are 7.37% and 11.94% respectively. These differences in settlement are observed because the model suggested by Ghosh ignores the vertical shear stress transfer at the fill-geosynthetic interface which has appropriately been incorporated in the present model along with the horizontal shear stress transfer. The consideration of vertical shear stress transfer at the fill-geosynthetic interface is more logical especially at higher load intensities for which membrane effect of reinforcement is more.

Figure 5.5 shows the effect of load intensity on the mobilised tensile force distribution in the geosynthetic reinforcement as obtained by the present study and as computed by using the model suggested by Ghosh (1991) in similar situations. Firstly, it is observed that for any load intensity, the mobilised tensile force is maximum and will be finite at the centre of the loaded region (the formulation presented can not determine the exact value due to computational problems) and it decreases rapidly up to the edge of the loaded region beyond which the rate of decrease is almost linear. At the edge of the reinforcement, the mobilised tensile force is zero because of one of the assumptions that the frictional resistance mobilised over the length of the geosynthetic reinforcement is sufficient to balance the tensile forces in it. Secondly, it is noted that for smaller load intensity ($q^* = 0.1$), the mobilised tensile force distribution predicted by the present model is almost similar to that predicted by Ghosh model whereas for higher load intensities the present model predicts relatively less mobilised tensile force at any location. For example,

at the edge of the loaded region the mobilised tensile force predicted by the present model is relatively less by 21.03% for $q^* = 0.5$ whereas for $q^* = 1.0$, the corresponding value is 34.42%. The mobilised tensile forces as predicted by the present study are consistent with the settlement predictions shown in Fig. 5.4.

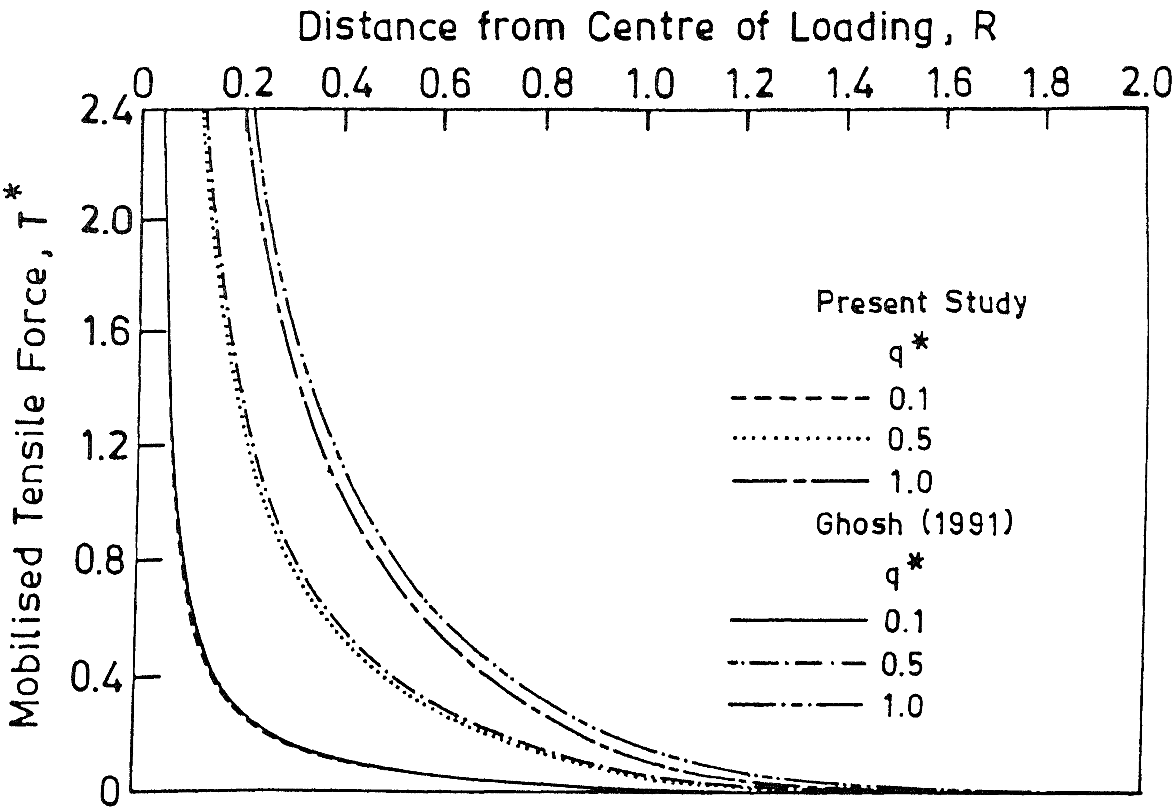


Fig. 5.5. Mobilised tensile force distribution profiles - comparison of foundation models.

5.5 DISCUSSION OF RESULTS

5.5.1 Effect of Lateral Stress Ratio

Throughout the study on the effect of lateral stress ratio in the granular fill on the settlement characteristics of geosynthetic-reinforced granular fill - soft soil system, $T_p^* = 0.0$, $\alpha = 1$ and $U = 100\%$ are considered in the present model.

Figure 5.6 shows typical load-settlement curves both at the centre and at the edge of

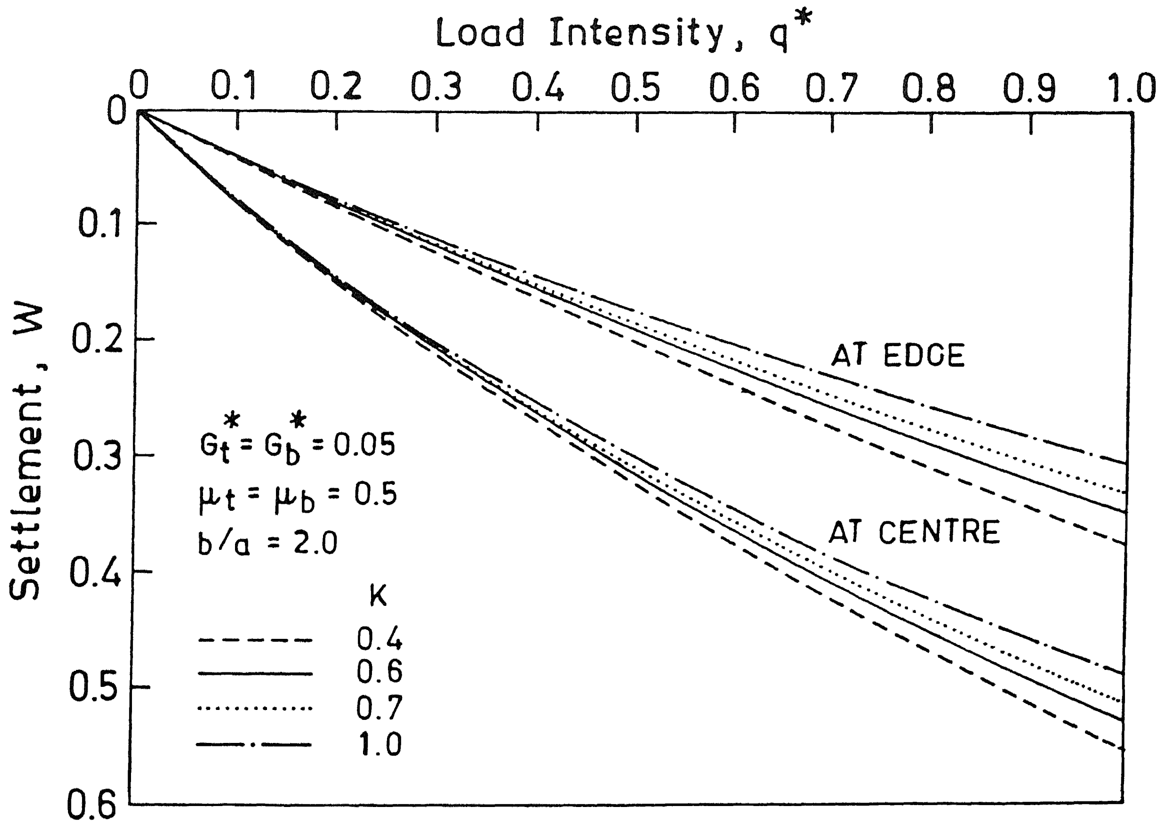


Fig. 5.6. Load-settlement curves - effect of lateral stress ratio in the granular fill.

the loaded region for various lateral stress ratios in the granular fill. It is noticed that at much lower load intensities, no appreciable reductions are noticed at both the centre and

the edge with the increase of K. However, significant settlement reductions occur at higher load intensities. For example, as K increases from 0.4 to 1.0, the settlement reduces by 1.61 %, 6.83%, and 11.89% at the centre of the loaded region for $q^* = 0.1$, 0.5, and 1.0 respectively whereas, the corresponding settlement reductions at the edge are 4.20%, 12.93%, and 18.62% respectively. Thus, one can reduce the settlement of the geosynthetic-reinforced granular fill - soft soil system significantly by increasing lateral stress ratio in the granular fill.

Figure 5.7 shows typical settlement profiles for different shear parameters of the granular fill, bringing out the effect of lateral stress ratio in the granular fill. It is observed

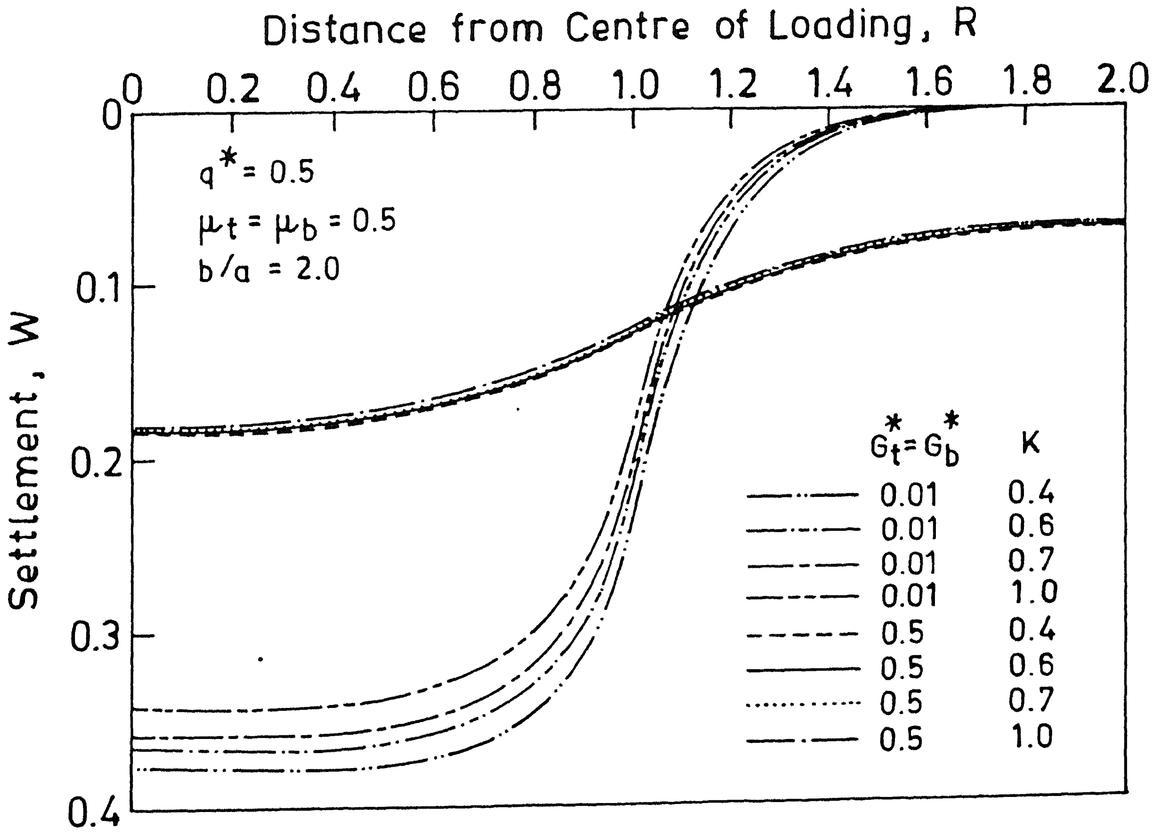


Fig. 5.7. Settlement profiles - effect of lateral stress ratio in the granular fill at its different shear parameters.

that an increase of K results in significant settlement reduction for lower shear parameters of the granular fill. For example, as K increases from 0.4 to 1.0, the settlement at the centre of the loaded region reduces by 8.8% for $G_t^* = G_b^* = 0.01$ whereas, the settlement reduction is 2.70% for $G_t^* = G_b^* = 1.0$. The corresponding reductions at the edge of the loaded region are 20.83% and 3.85% respectively.

Figure 5.8 shows typical settlement profiles for different interfacial friction coefficients, bringing out the effect of lateral stress ratio in the granular fill. It is observed that the settlement reduces significantly with the increase of K for higher interfacial friction coefficients whereas, for lower friction coefficients the settlement reductions are relatively

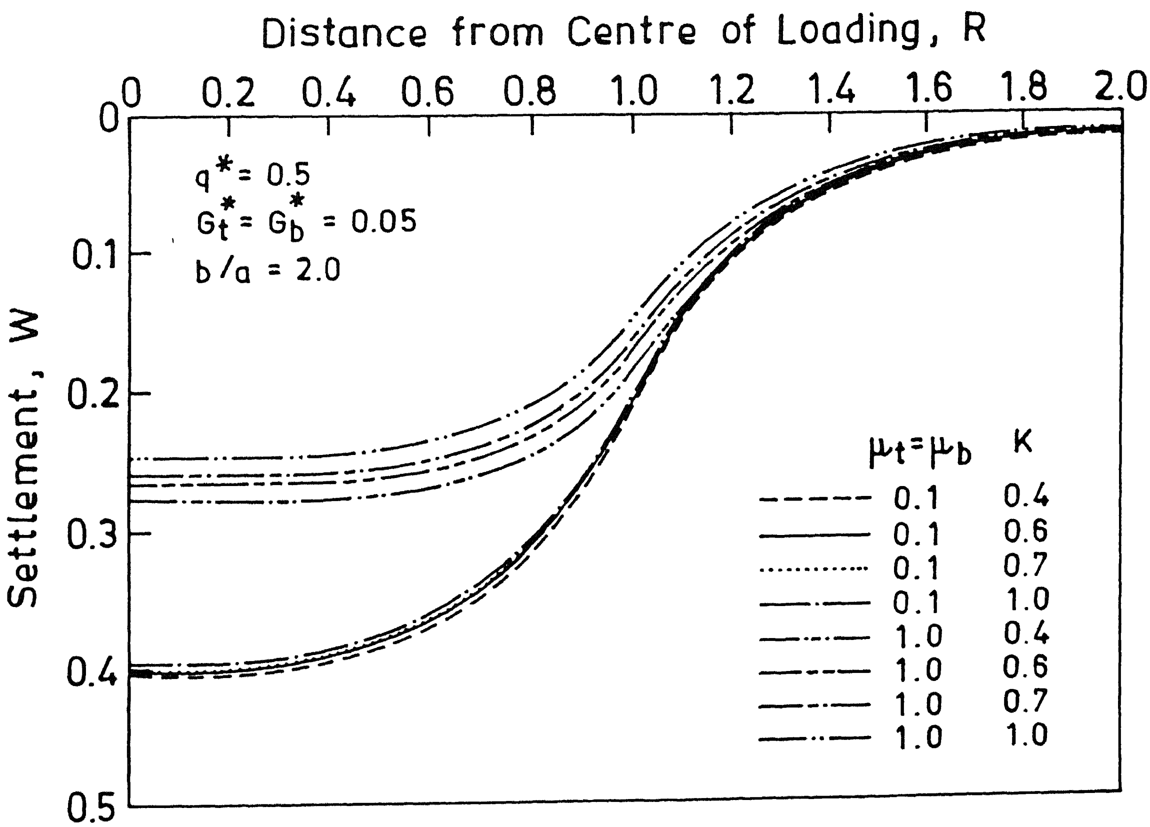


Fig. 5.8. Settlement profiles - effect of lateral stress ratio in the granular fill at different interfacial friction coefficients.

low. For example, as K increases from 0.4 to 1.0, the settlement at the centre of the loaded region reduces by 2.22% for $\mu_t = \mu_b = 0.1$ and by 10.87% for $\mu_t = \mu_b = 1.0$ whereas, the corresponding settlement reductions at the edge of the loaded region are 4.19% and 19.15% respectively.

5.5.2 Effect of Prestressing the Geosynthetic Reinforcement

Throughout the study on the effect of prestressing the geosynthetic reinforcement on the settlement characteristics of geosynthetic-reinforced granular fill - soft soil system, $\alpha = 1$, and U = 100% are considered in the present model.

Figure 5.9 shows typical load-settlement curves both at the centre and at the edge of the loaded region for various values of prestress in the geosynthetic reinforcement. It is observed that as a result of prestressing the geosynthetic reinforcement, the settlement reduces both at the centre and at the edge of the loaded region. For any value of prestress, it is noted that the settlement reduction is more at the centre than that at the edge of the loaded region. For example, for $T_p^* = 0.5$, the reduction in settlement at the centre of the loaded region is 45.95% for $q^* = 1.0$ whereas, reduction is 35.64 % at the edge. This indicates that as a result of prestressing both the total and differential settlements of the loaded region decreases significantly. If we compare these reductions in settlement at the centre and at the edge for $T_p^* = 0.5$ and $q^* = 0.1$ which are 59.11% and 39.86% respectively, it can be noticed that the order of settlement reduction for any value of prestress in the geosynthetic reinforcement is relatively more for lower load intensities than that for higher load intensities.

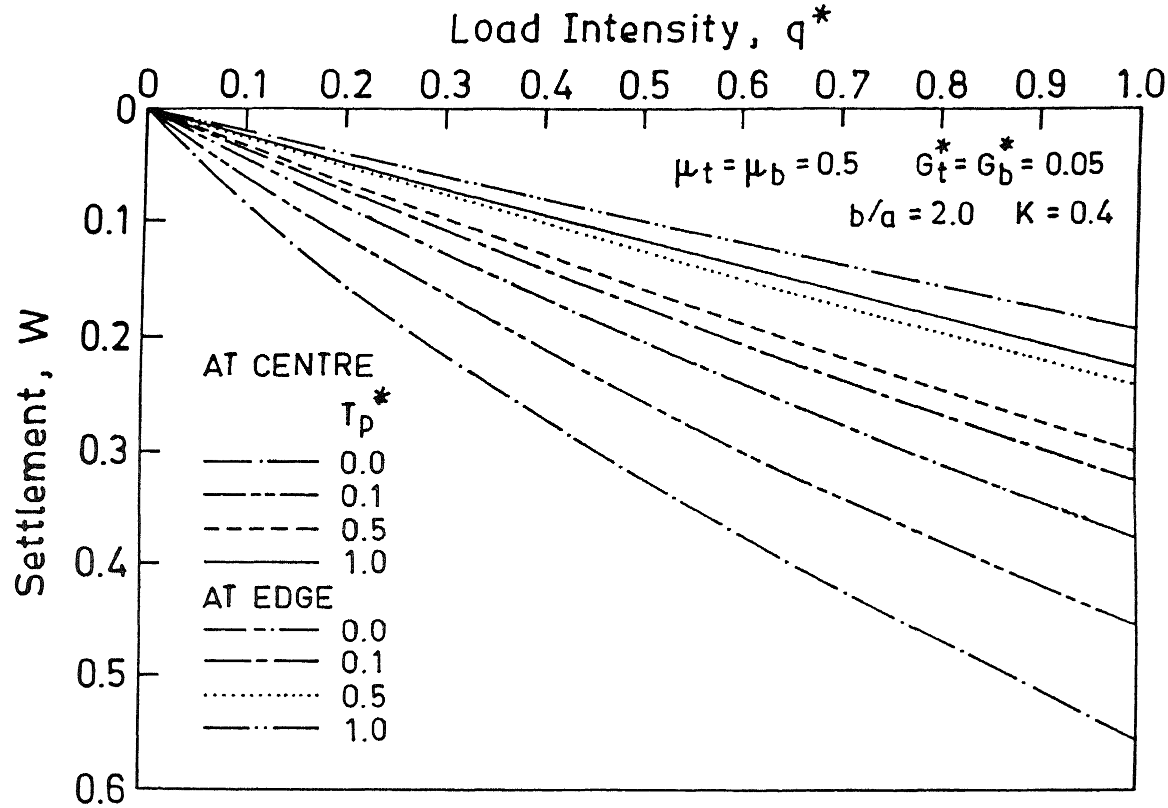


Fig. 5.9. Load-settlement curves for various values of prestress.

Figure 5.10 shows typical settlement profiles for different values of nondimensional shear parameter of the granular fill. It is noticed that the settlement reduces significantly within the loaded region as a result of prestressing the geosynthetic reinforcement for lower value of shear parameter whereas for higher value of shear parameter, settlement reductions are relatively quite less. For example, for $T_p^* = 0.1$, the reduction in settlement is 28.80% at the centre and 23.33 % at the edge of the loaded region for $G_t^* = G_b^* = 0.01$ whereas, the corresponding reductions are 6.52% and 4.33% respectively for $G_t^* = G_b^* = 1$.

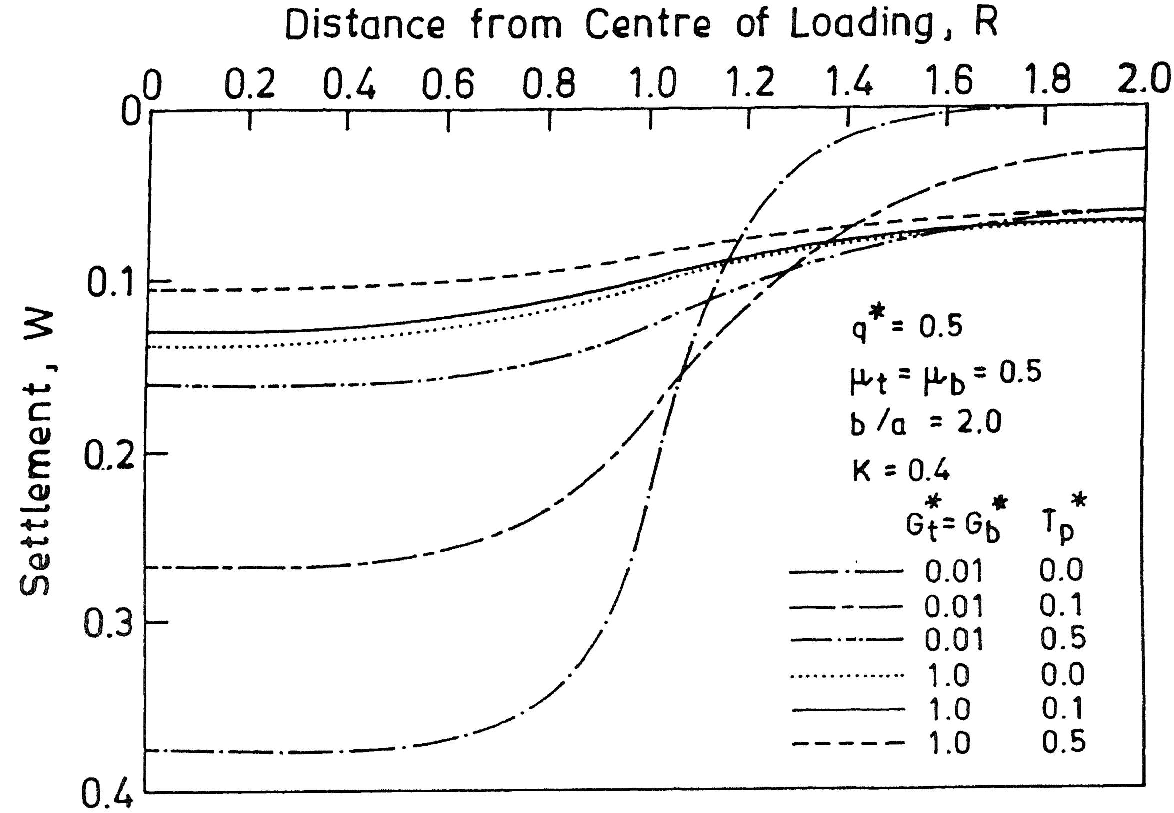


Fig. 5.10. Settlement profiles - effect of prestressing for different shear parameters of the granular fill.

Figure 5.11 shows typical settlement profiles for different interfacial friction coefficients, bringing out the effect of prestressing the geosynthetic reinforcement. It is noted that for $T_p^* = 0.1$, the settlement reduction is 29.13% at the centre and 14.42% at the edge of the loaded region for $\mu_t = \mu_b = 0.1$ whereas, for $\mu_t = \mu_b = 1.0$, the corresponding reductions in settlement are 18.12% and 13.83% respectively. These results indicate that prestressing the geosynthetic reinforcement brings out relatively more reduction in settlement within the loaded region for lower value of interfacial friction coefficient compared with the reduction for higher value of interfacial friction coefficient.

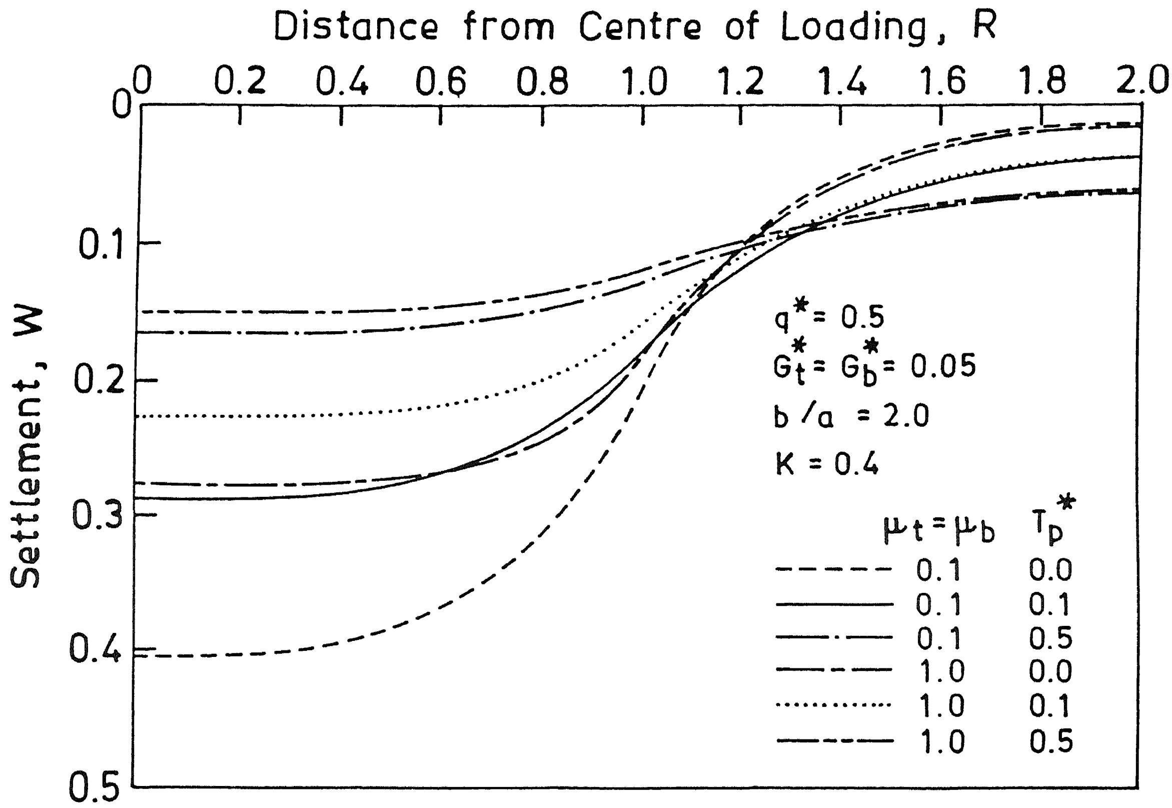


Fig. 5.11. Settlement profiles - effect of prestressing for different interfacial friction coefficients.

Figure 5.12 shows typical settlement profiles for different widths of reinforced zone, bringing out the effect of prestressing. It is observed that the reduction in settlement at any location within the loaded region due to prestress in the geosynthetic reinforcement is more for higher width of reinforced zone than that for lower width of reinforced zone. For example, for $T_p^* = 0.1$, the reduction in settlement at the centre of the loaded region is 7.5% for b/a = 1.2 whereas, the corresponding reduction in settlement for b/a = 2.0 is 21.43%.

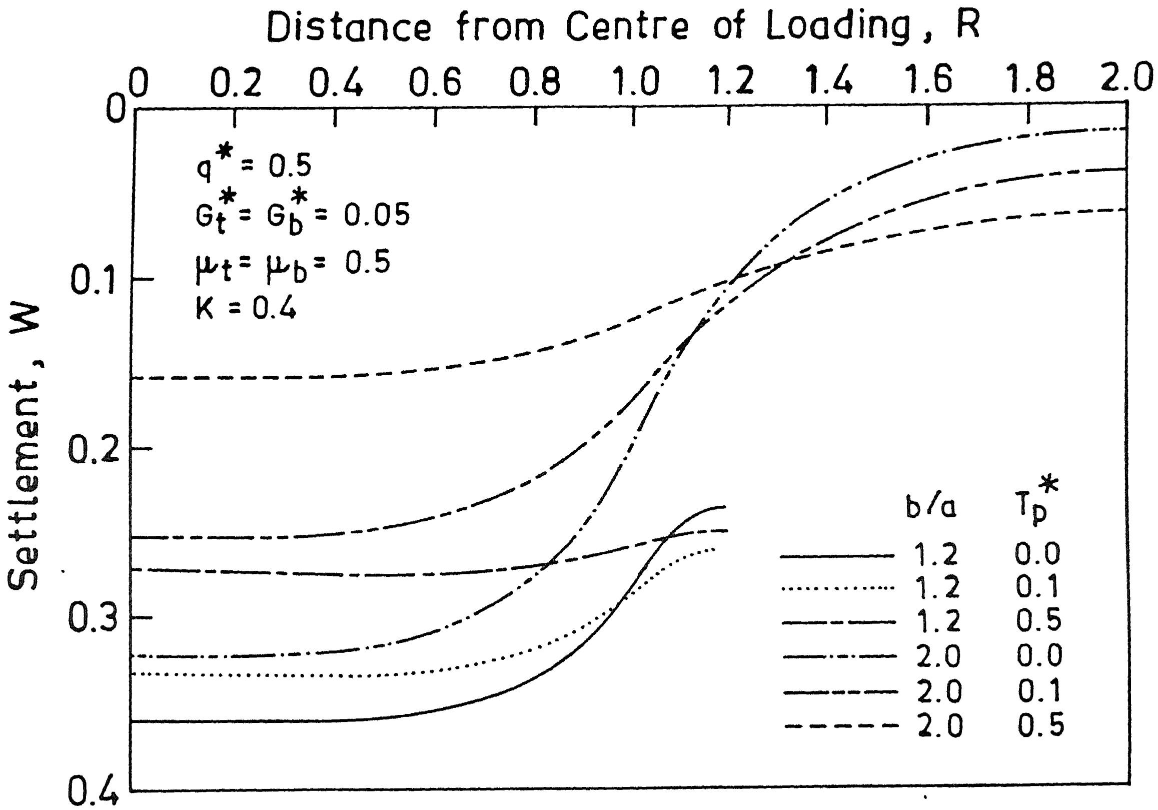


Fig. 5.12. Settlement profiles - effect of prestressing for different widths of reinforced zone.

Figure 5.13 shows the typical settlement profiles for different lateral stress ratio, bringing out the effect of prestressing on the settlement response. It is noticed that for low prestress, the settlement reduction at any location due to increase in lateral stress ratio in the granular fill is more than the case where high prestress is used. For example, as K increases from 0.4 to 1.0, the settlement reduces by 4.74 % at the centre of the loaded region and by 7.51% at the edge for $T_p^* = 0.1$ whereas, the corresponding reductions in settlement are 2.53% and 2.4% respectively for $T_p^* = 0.5$.

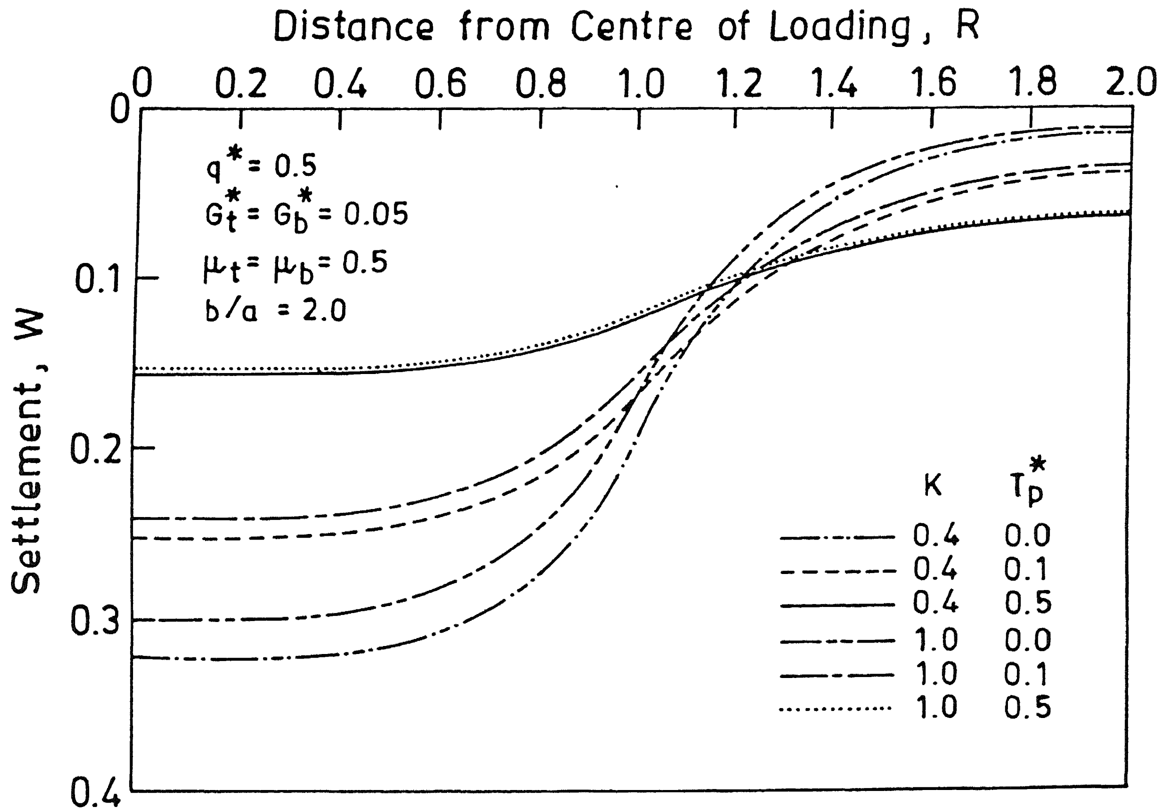


Fig.5.13. Settlement profiles - effect of prestressing for different lateral stress ratios .

Figure 5.14. shows the effect of prestressing on the mobilised tensile force distribution profiles. It is noticed that prestressing the geosynthetic reinforcement affects the mobilisation of tensile force in the geosynthetic reinforcement. With the increase in prestress, the mobilised tensile force in the geosynthetic reinforcement at any location increases. The mobilised tensile force is zero at R = 2.0 because of one of the boundary conditions assumed.

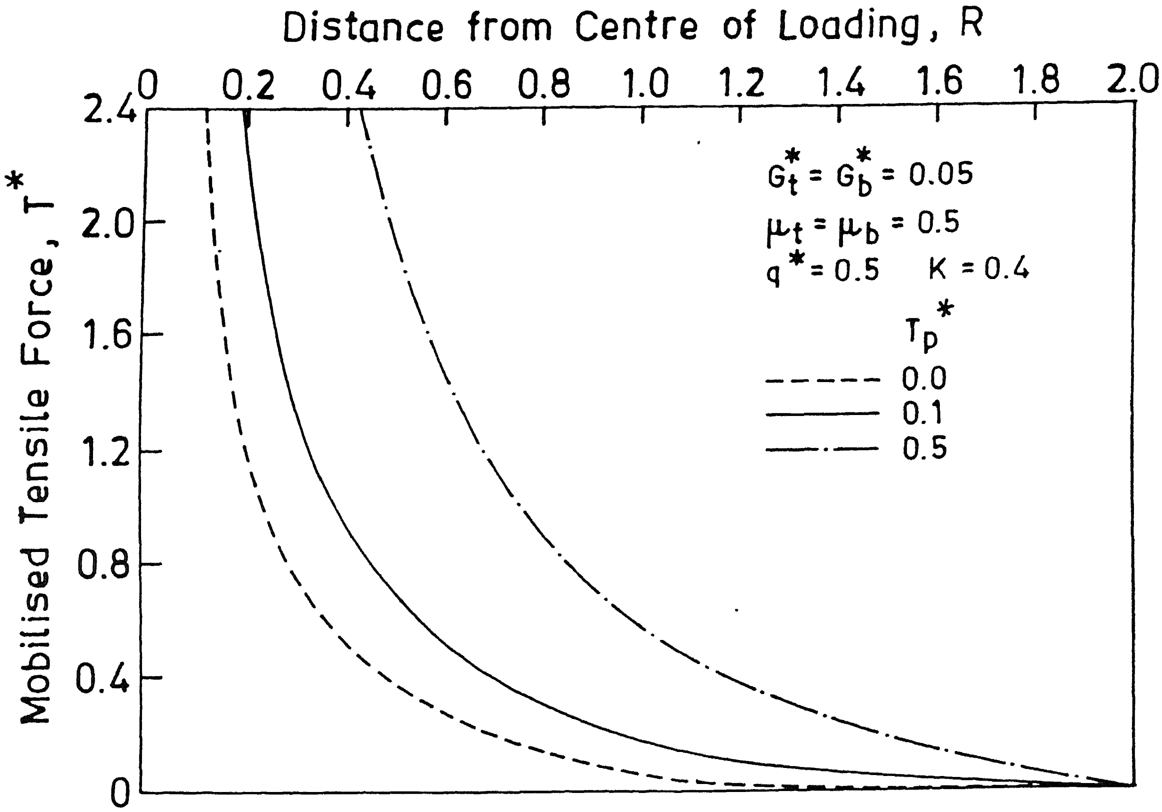


Fig. 5.14. Mobilised tensile force distribution profiles - effect of prestressing the geosynthetic reinforcement.

5.5.3 Effect of Compressibility of the Granular Fill

Throughout the study on the effect of compressibility of the granular fill on the settlement characteristics of geosynthetic-reinforced granular fill - soft soil system, U = 100% is considered in the present model. During the study increasing values of modular ratio, α, are considered. As the value of α increases, the relative compressibility of the granular fill decreases and when it approaches infinity, the granular fill becomes incompressible.

Figure 5.15 shows the effect of compressibility of the granular fill on the load-settlement behaviour of geosynthetic-reinforced granular fill - soft soil system at the

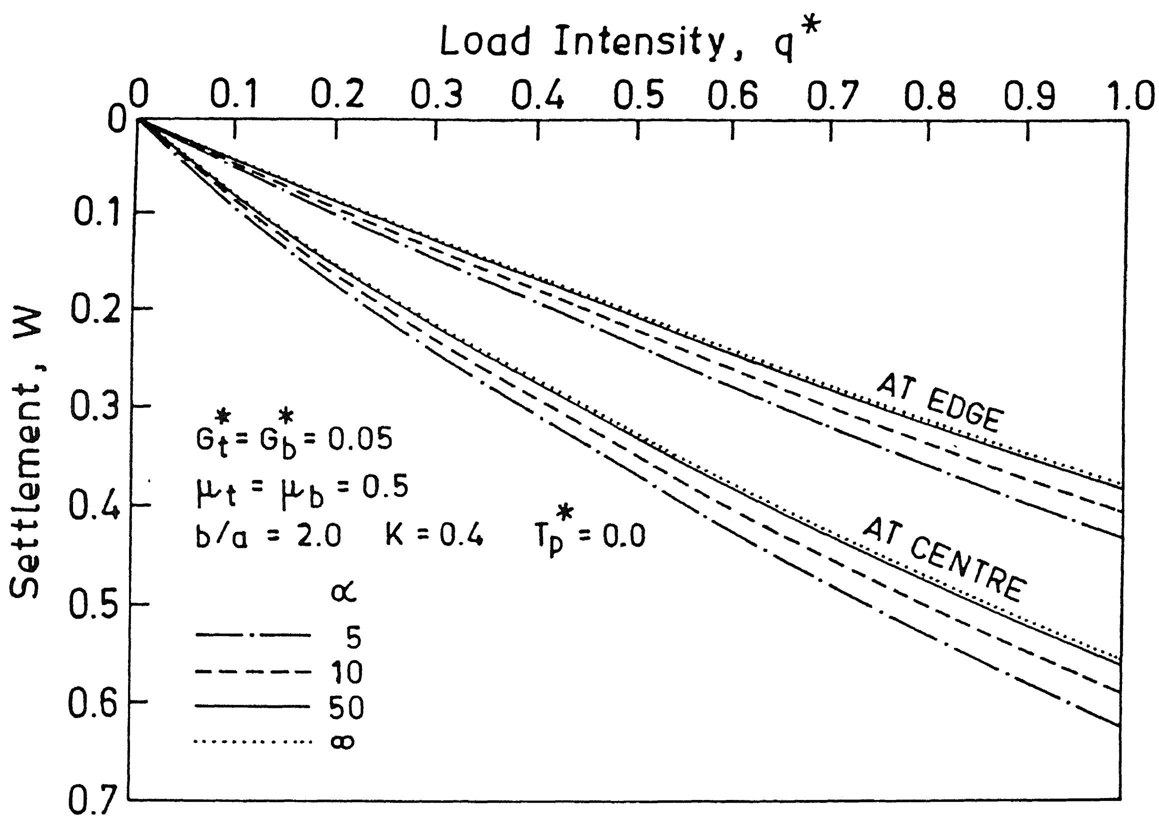


Fig. 5.15. Load-settlement curves - effect of compressibility of the granular fill.

centre and at the edge of the loaded region for typical values of parameters. It is observed that for any increase in the value of α, resulting in decrease of compressibility of the granular fill, the order of decrease in settlement is more at the edge than that at the centre of the loaded region. For example, for $q^* = 1.0$, as α increases from 5 to 50, the settlement reduces by 9.94% at the centre and by 11.36% at the edge of the loaded region. For the increase of α from 5 to ∞, the corresponding reductions in settlement are 11.06% and 12.76% respectively. However, the settlement reductions are very small as the

value of α is increased. The comparison of the settlement reductions for the increase of α from 5 to 50 and from 5 to ∞ indicates that if α is more than 50, the consideration of the compressibility of the granular fill in settlement analysis may be ignored for routine field problems.

Figure 5.16 shows settlement profiles for different shear parameters of the granular fill, bringing out the effect of compressibility of the granular fill. It is observed that the settlement reduces with the increase of modular ratio, the reduction being more for lower shear parameter. For example, as α increases from 5 to 50, the settlement reduces by 11.19% at the centre and by 12.54% at the edge of the loaded region for $G_t^* = G_b^* = 0.01$

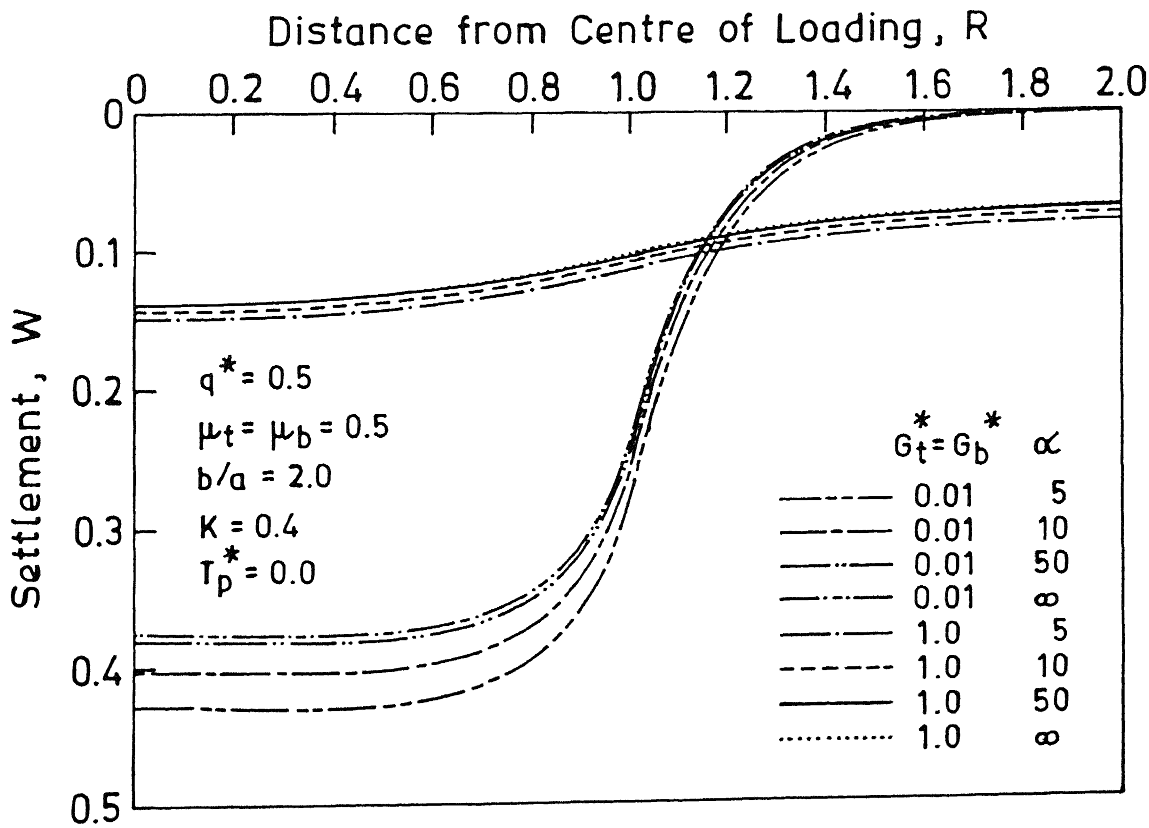


Fig. 5.16. Settlement profiles - effect of compressibility of the granular fill for its different shear parameters.

whereas, the corresponding reductions are 7.33% and 8.70% respectively for $G_t^* = G_b^* = 1.0$.

Figure 5.17. shows settlement profiles for different interfacial friction coefficients, bringing out the effect of compressibility of the granular fill. It is noticed that an increase in the value of α results in significant reduction in settlement, the reduction being relatively more for lower interfacial friction coefficient. For example, as α increases from

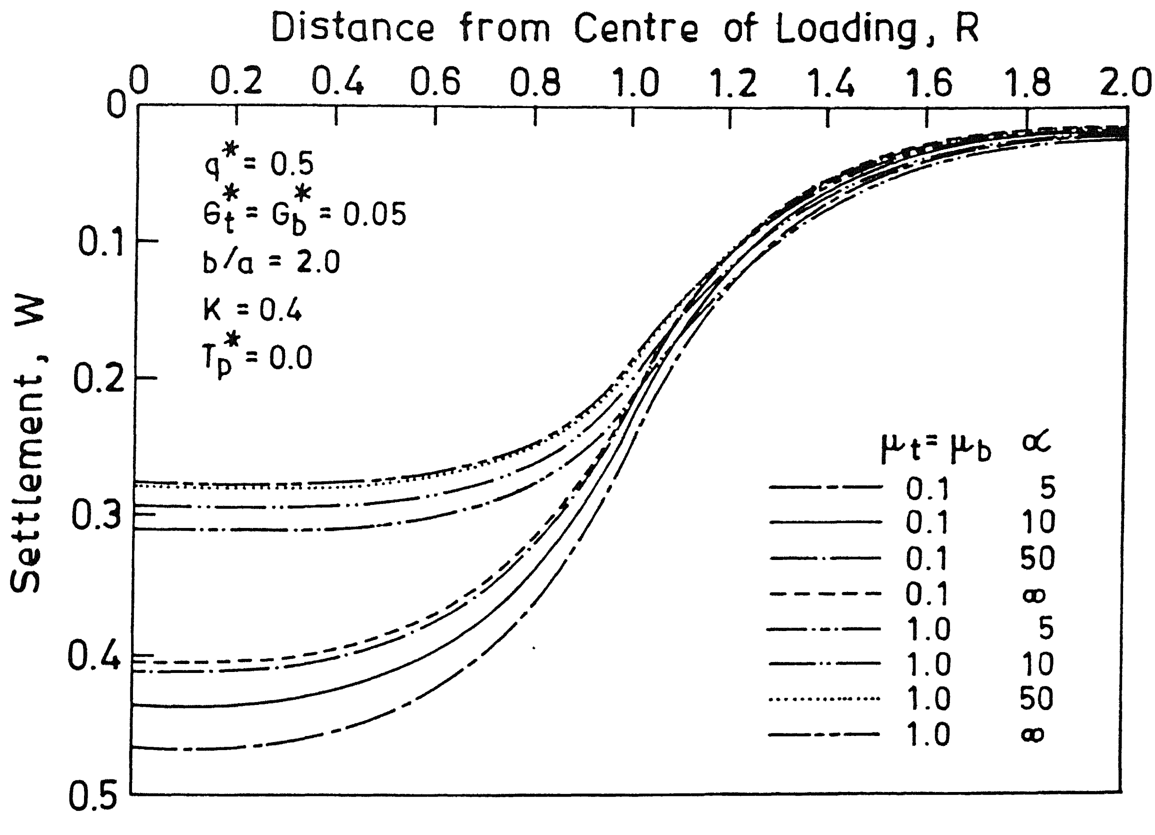


Fig. 5.17. Settlement profiles - effect of compressibility of the granular fill for different interfacial friction coefficients.

5 to 50, the settlement reduces by 11.80% at the centre and by 13.15% at the edge of the loaded region for $\mu_t = \mu_b = 0.1$ whereas, the corresponding reductions in settlement are 9.68% and 11.16% respectively for $\mu_t = \mu_b = 1.0$.

Figure 5.18 shows the effect of compressibility of the granular fill on the settlement behaviour of the reinforced soil system in case of prestress in the geosynthetic reinforcement. It is observed that the increase of α results in reduction in settlement, the

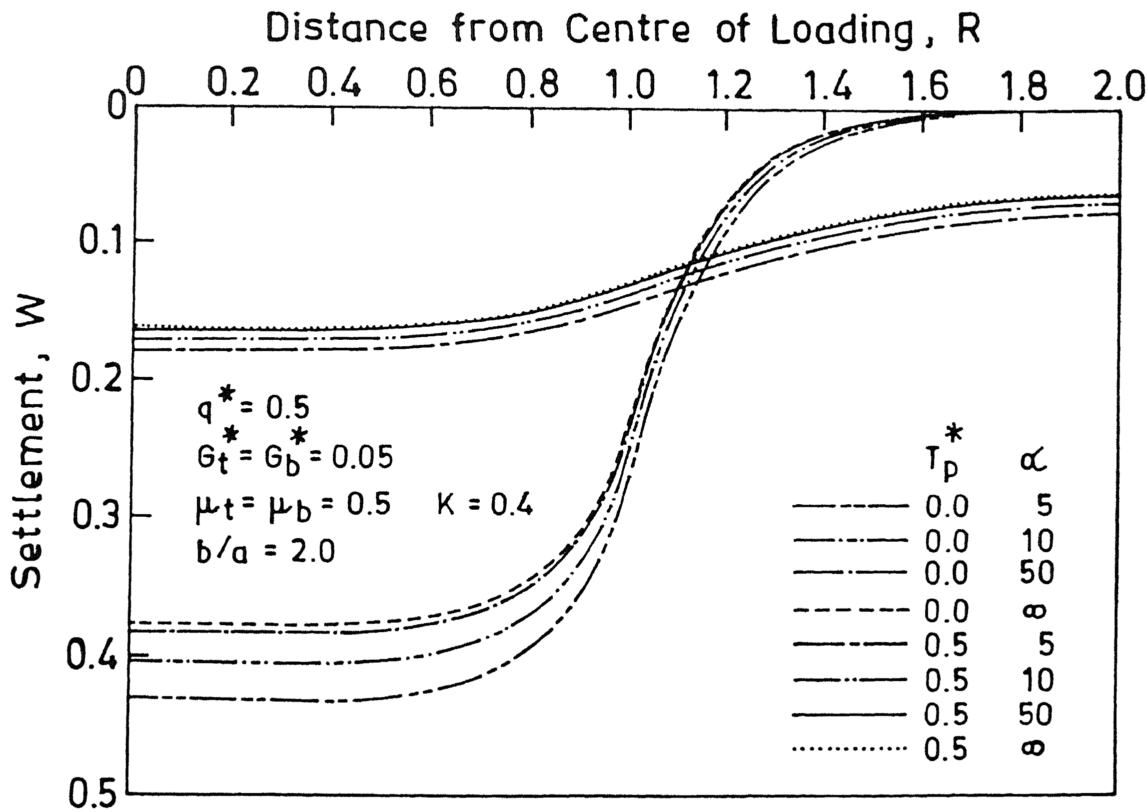


Fig. 5.18. Settlement profiles - effect of compressibility of granular fill in case of prestress in the geosynthetic reinforcement.

reduction being relatively more for the case of no prestress in the geosynthetic reinforcement. For example, as α increases from 5 to 50, the settlement reduces by 11.19% at the centre and by 12.54% at the edge of the loaded region in case of no prestress in the reinforcement. For $T_p^* = 0.5$, the corresponding reductions in settlement are 8.43% and 9.72% respectively.

5.5.4 Time-Dependent Behaviour

The results presented below reveal the effect of several parameters on the settlement response of the geosynthetic-reinforced granular fill - soft soil system at various stages of consolidation of the of the soft foundation soil.

Figure 5.19 shows the variation of the settlement at the centre of the loaded region with degree of consolidation for various load intensities. It is observed that for very smaller load, the settlement increases almost linearly with the increase of degree of consolidation whereas, for higher loads, the rate of increase is nonlinear and decreases as the degree of consolidation increases. For example, for $q^* = 0.5$, the increase in settlement is 0.078 for the increase of U from 10% to 20% whereas, the increase is 0.047 for the increase of U from 50% to 60%.

Figure 5.20 shows settlement profiles for different nondimensional shear parameters of granular fill at various stages of consolidation of the soft foundation soil. It is noticed that for any increase of degree of consolidation, the increase in settlement at any location within the loaded region is more for lower shear parameter than that for

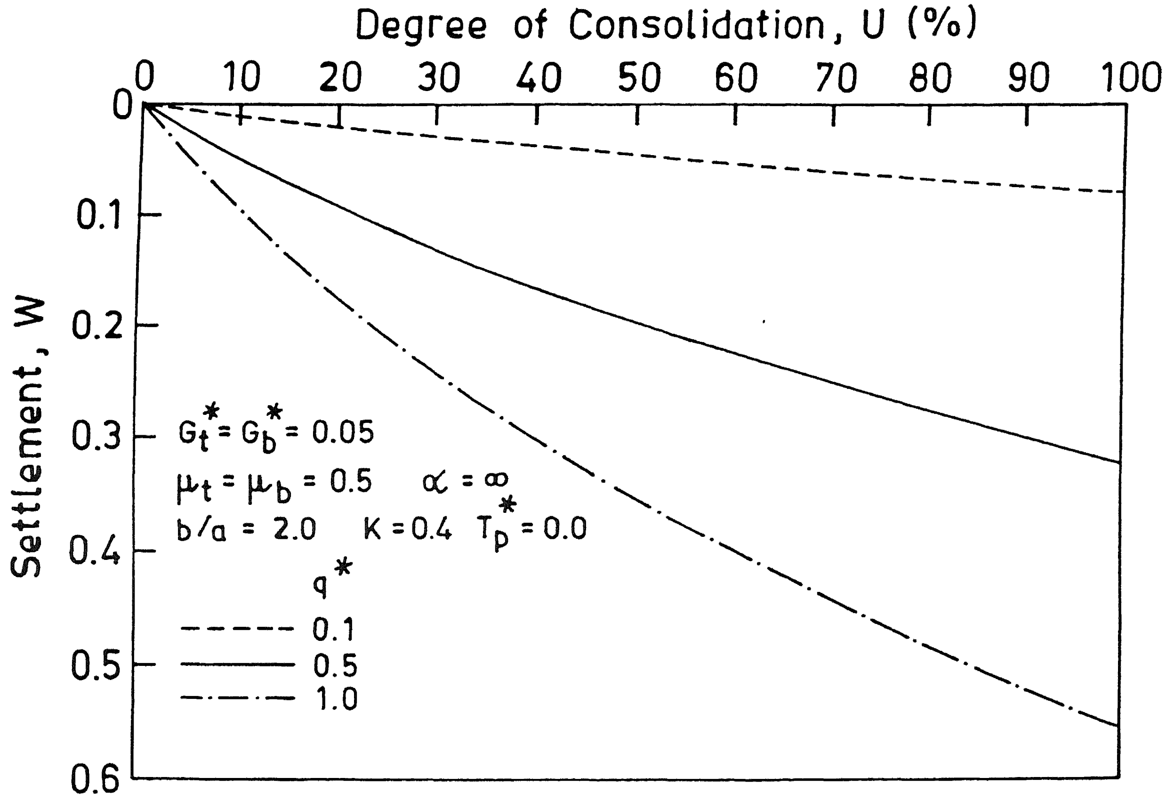


Fig. 5.19. Variation of settlement at the centre of the loaded region with degree of consolidation of soft foundation soil for various load intensities.

higher shear parameter. For example, as U increases for 50% to 90%, the settlement at the centre of the loaded region increases by 60.19% for $G_t^* = G_b^* = 0.01$ and by 34.77% for $G_t^* = G_b^* = 1.0$.

Figure 5.21. shows settlement profiles for different interfacial friction coefficients at various stages of consolidation of soft foundation soil. It is observed that settlement profiles are almost similar during initial stages of consolidation. However, in later stages of consolidation, for any increase of degree of consolidation, the increase in settlement under

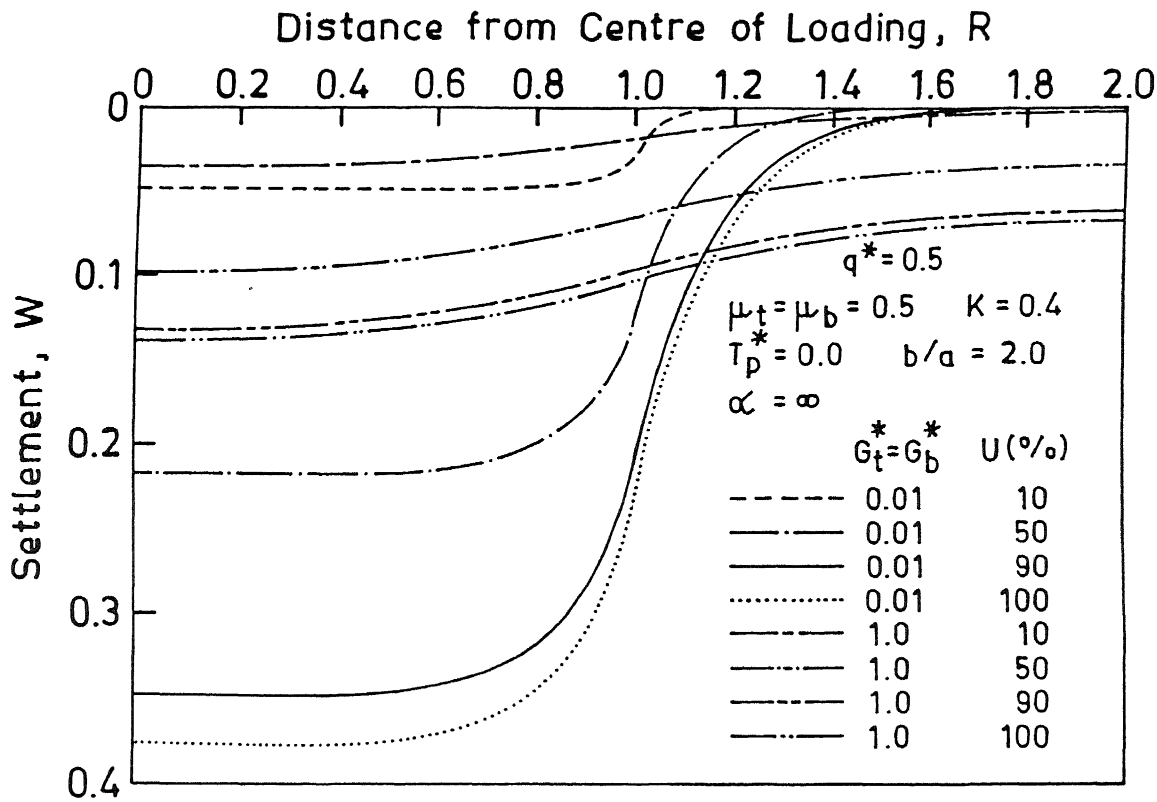


Fig. 5.20. Settlement profiles for different shear parameters of granular fill at various stages of consolidation of the soft foundation soil.

the loaded region is relatively more for lower interface friction coefficient whereas beyond the loaded region the trend is reversed. For example, as U increases from 50% to 90%, the settlement at the centre of the loaded region increases by 64.31% for $\mu_t = \mu_b = 0.1$ and by 49.13% for $\mu_t = \mu_b = 1.0$.

Figure 5.22 shows settlement profiles for different widths of reinforced zone at various stages of consolidation of soft foundation soil. It is noticed that during the initial stages of consolidation of soft foundation soil the settlement profiles for both widths of reinforced

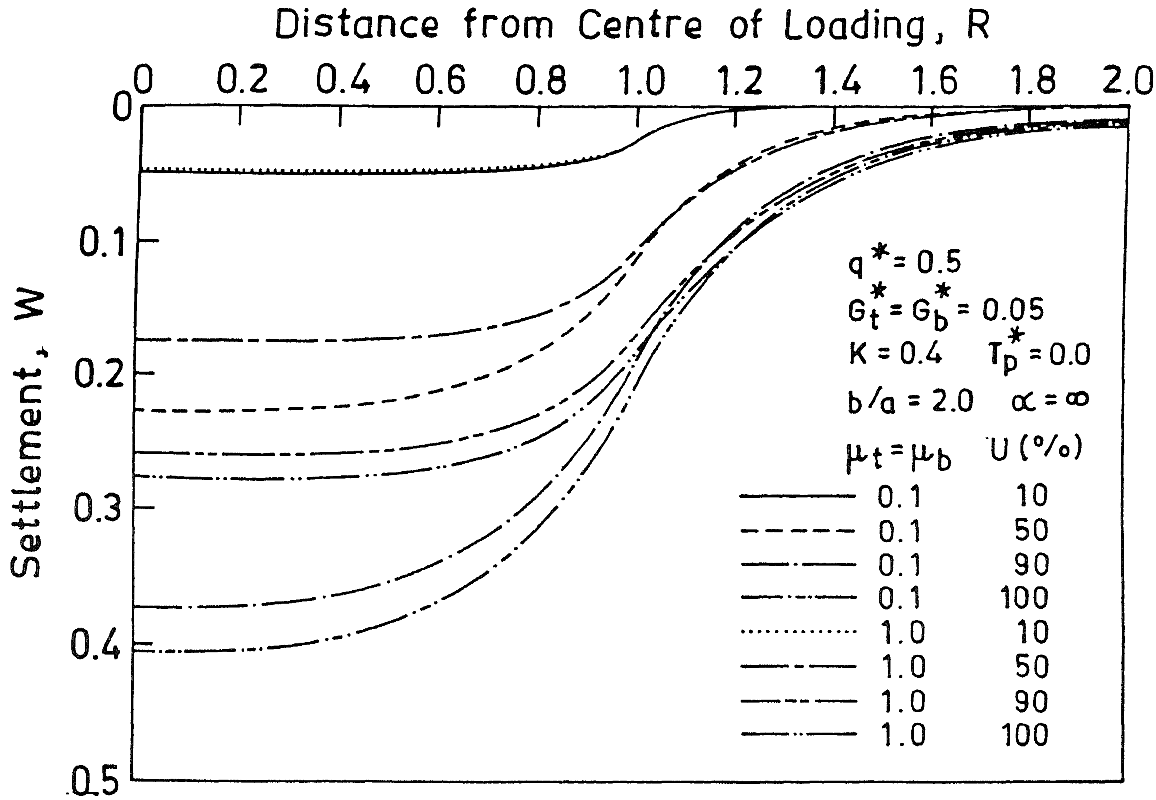


Fig. 5.21. Settlement profiles for different interfacial friction coefficients at various stages of consolidation of the soft foundation soil.

zone are almost identical. However, during later stages of consolidation, for any increase of degree of consolidation, the increase in settlement at any location is more for lower width of reinforced zone than that for higher width of reinforced zone. For example, as U increases from 50% to 90%, the settlement increases by 61.76% for b/a = 1.2 and by 52.55% for b/a = 2.0.

Figure 5.23 shows settlement profiles for different lateral stress ratio in the granular fill at various stages of consolidation of soft foundation soil. It is observed that during initial stages of consolidation, the settlement profiles are identical for both values of K.

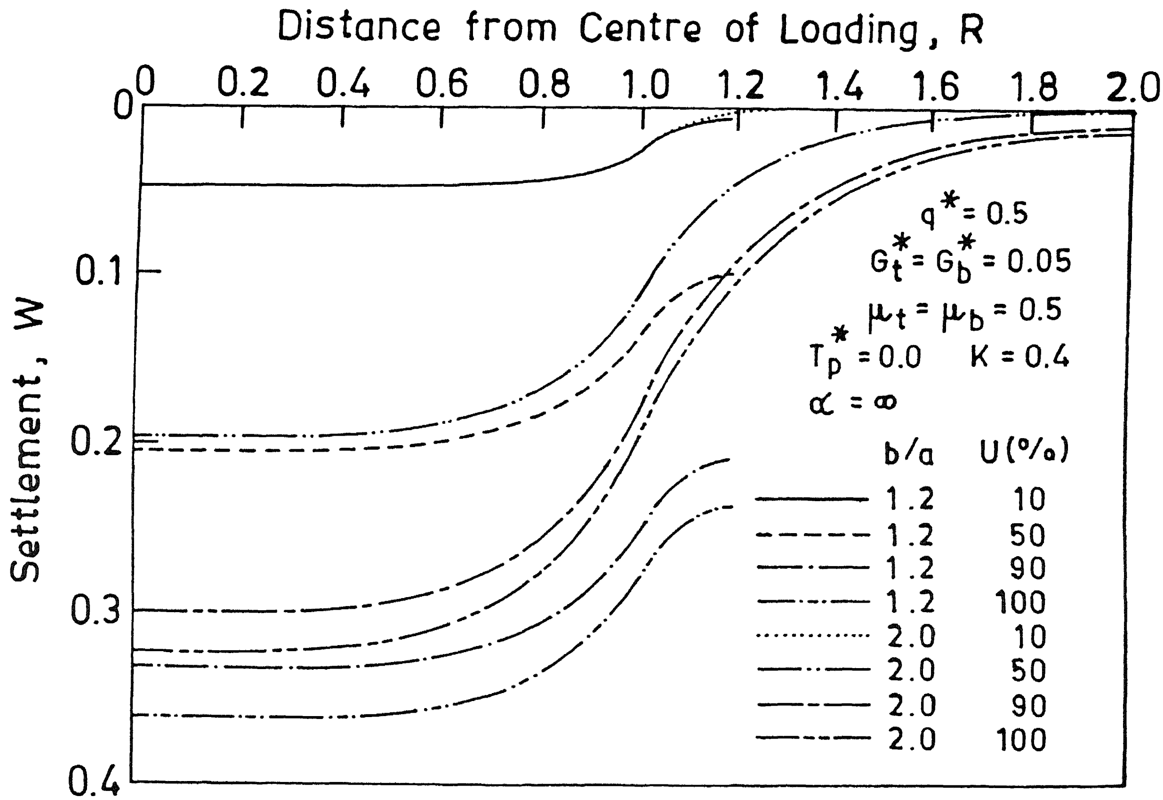


Fig. 5.22. Settlement profiles for different widths of reinforced zone at various stages of consolidation of the soft foundation soil.

However, during later stages of consolidation, for any increase of degree of consolidation, the increase in settlement at the centre of the loaded region is more for lower value of K. For example, as U increases from 50% to 90%, the settlement at the centre of the loaded region increases by 52.55% for K = 0.4 and by 50.26% for K = 0.7.

Figure 5.24 shows the effect of prestressing the geosynthetic reinforcement on the settlement profiles at different stages of consolidation of soft foundation soil. It is observed that for any increase of degree of consolidation, the increase in settlement at any location within the loaded region is relatively less for the case in which prestress is applied to geosynthetic reinforcement whereas, beyond the loaded region the trend is

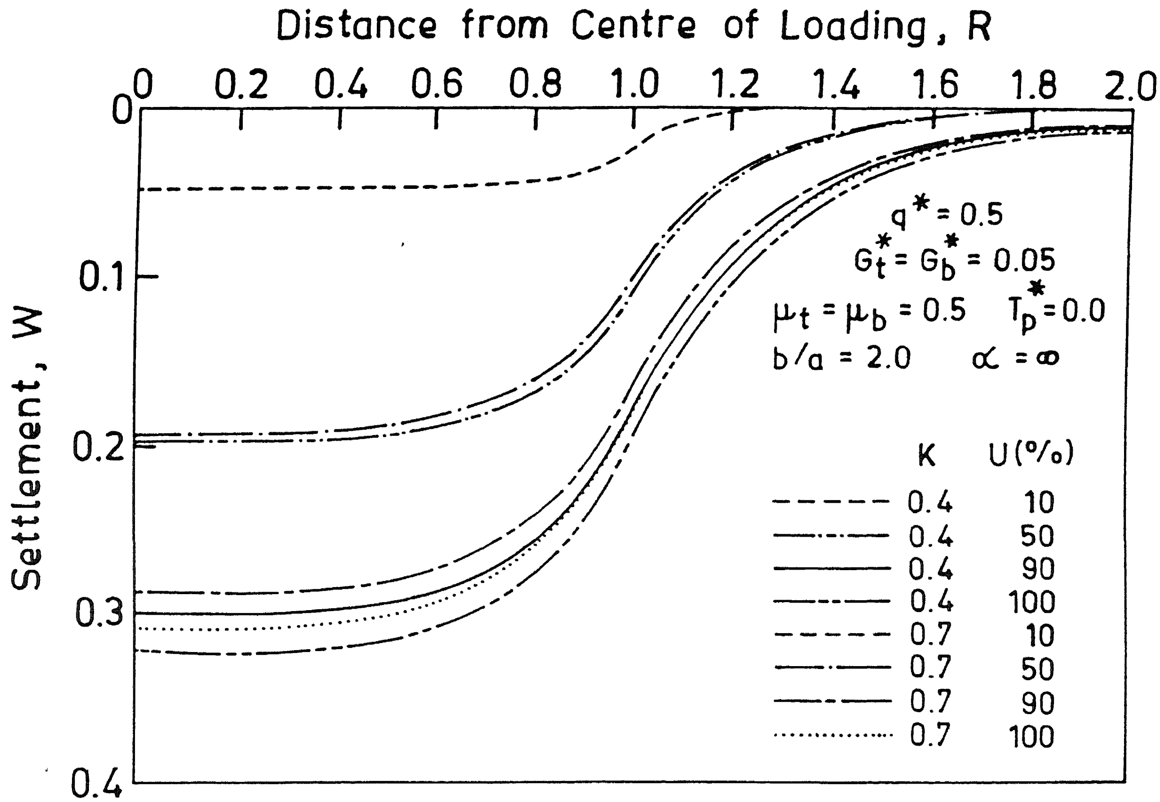


Fig. 5.23. Settlement profiles for different lateral stress ratios in the granular fill at various stages of consolidation of the soft foundation soil.

reversed. For example, as U increases from 50 to 90%, the settlement increases by 52.55% at the centre and by 62.28% at the edge of the loaded region for $T_p^* = 0.0$ whereas, the corresponding increments are 40.94% and 53.22% respectively for $T_p^* = 0.3$.

Figure 5.25 shows typical settlement profiles for different modular ratios at various stages of consolidation of soft foundation soil. It is observed that at any stage of consolidation of the soft foundation soil, the settlement at any location is more for lower value of α than that for higher value of α. For example, at 90% consolidation, the settlement at the centre of the loaded region is 0.344 for α = 5 and is 0.304 for α = 50.

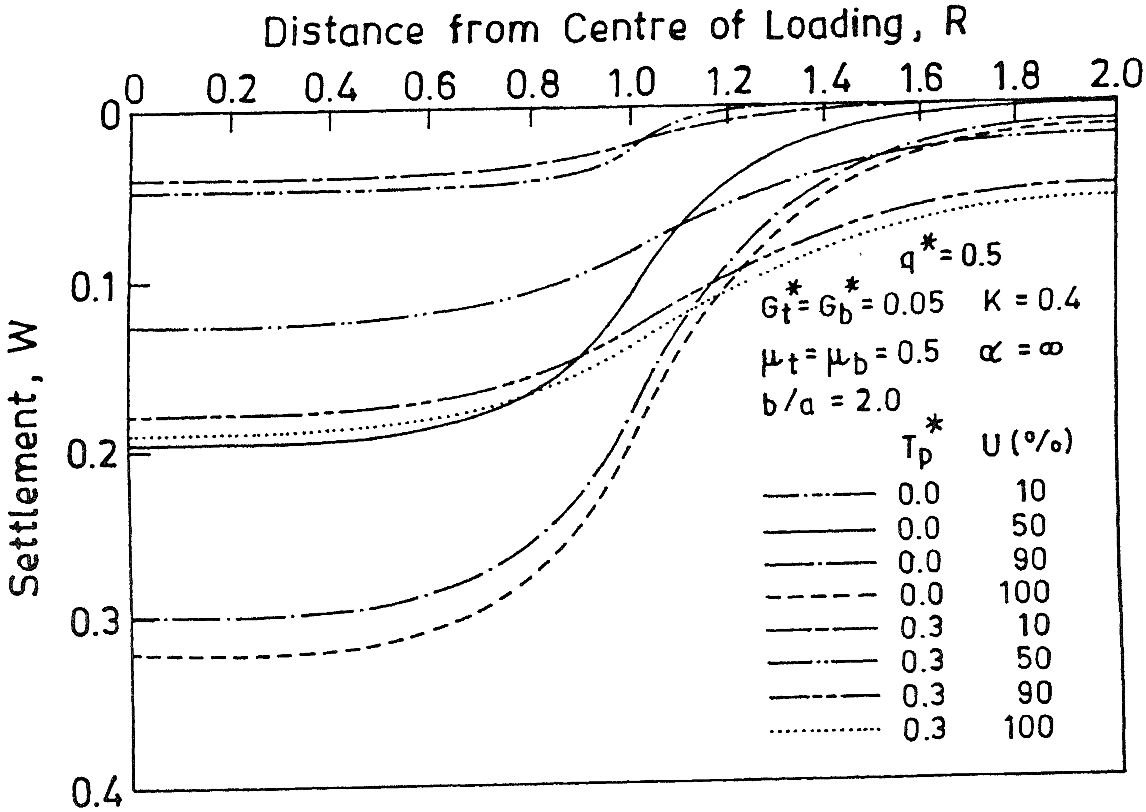


Fig. 5.24. Settlement profiles bringing out the effect of prestressing the geosynthetic reinforcement at various stages of consolidation of the soft foundation soil

5.6 CONCLUSIONS

The proposed foundation model has been employed successfully for the study of settlement characteristics of geosynthetic-reinforced granular fill - soft soil system in axi-symmetric conditions. The comparison of the settlement predictions by the present foundation model with those computed by using the exsiting foundation model have shown similar trend of results for common model parameters. The effects of lateral stress ratio in the granular fill, the prestressing of geosynthetic reinforcement, and the compressibility of the granular fill on the settlement behaviour of geosynthetic reinforced granular fill - soft soil system as well as its time-dependent behaviour have similar trends as observed in Chapter 3.

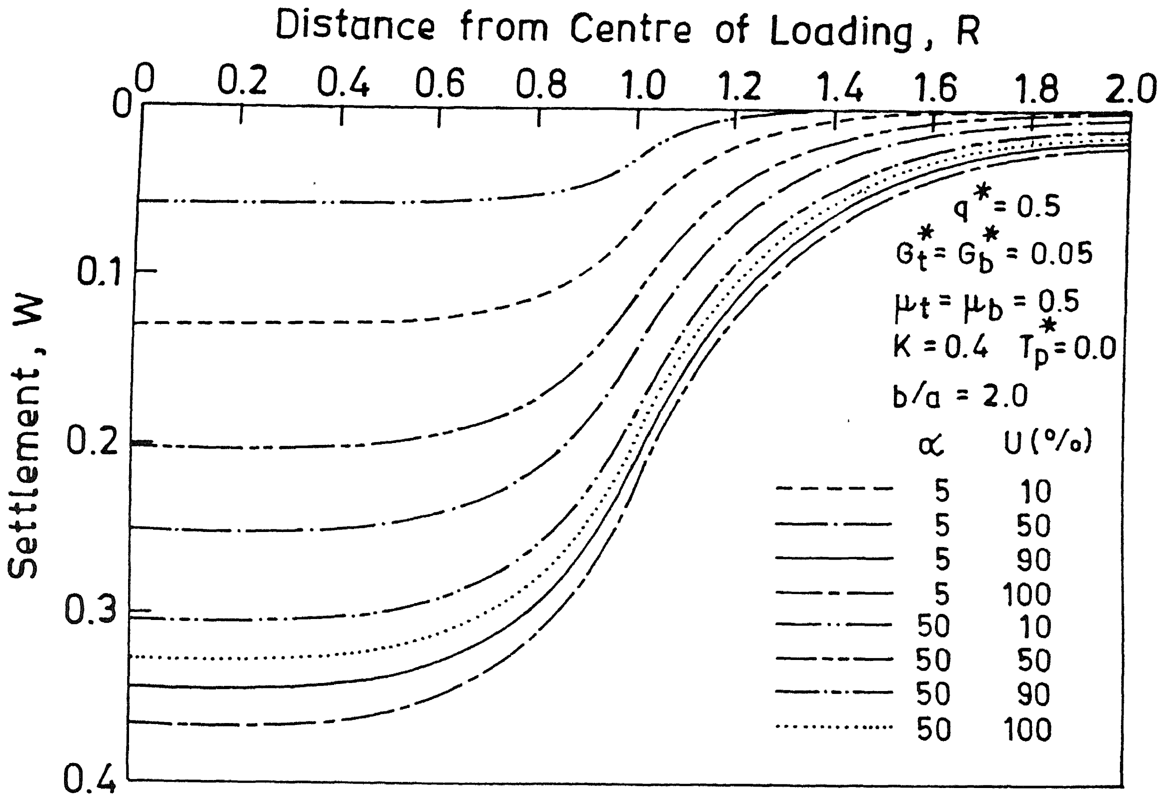


Fig. 5.25. Settlement profiles for different modular ratios at various stages of consolidation of the soft foundation soil.

CHAPTER 6

SUMMARY AND CONCLUSIONS

With the availability of competent geosynthetics, their uses in many applications have become more common and have proven to be an effective means of soil improvement. One such application is the reinforcement of a granular fill placed on a soft foundation soil. The geosynthetic reinforcement, mainly geotextiles and geogrids, is placed either at the interface of the granular fill and the soft subgrade or inside the granular fill. Such reinforced soil systems are used as foundations for shallow footings, embankments, unpaved roads, oil drilling platforms, heavy industrial equipments, parking lots etc.

From the analysis of the results of a large number of model tests conducted till very recently and the results presented through several analytical and numerical studies of geosynthetic-reinforced granular fill - soft soil system, it is observed that there are various aspects like vertical shear stress transfer at the fill-geosynthetic interface under large deformations, the prestressing of the geosynthetic reinforcement, the compressibility of the granular fill, and the time-dependent behaviour resulting from consolidation of the soft foundation soil, which need their considerations while estimating the settlement of the reinforced soil system under the applied load and using it as a foundation in the field.

In the present work, a mechanical foundation model is developed incorporating the factors as stated above. The proposed foundation model idealises the behaviour of each sub-system of the reinforced granular fill - soft soil system by mechanical elements such as Winkler springs, Pasternak shear layer, rough elastic membrane and dashpots commonly used to solve many soil-foundation-structure interaction problems in geotechnical engineering. Considering the equilibrium of different elements under the applied load, the equations governing the response function of the model are derived separately for both plane strain and axi-symmetric situations existing in many field problems. During derivations for the response function, some assumptions are made to make the model simple so that one can understand and use it very easily for solving field problems.

The parametric studies are carried out to bring out the effects of several parameters on the settlement response of the model. The governing equations in their nondimensional forms are solved iteratively by finite difference method and the results are presented in nondimensional form. Based on the results and discussion presented for plane strain and axi-symmetric problems, the following generalised conclusions are drawn:

1. The proposed foundation model for geosynthetic-reinforced granular fill - soft soil system is well suited to evaluate the settlement response over a large range of various governing parameters and may be used in situations of large deformations as well as in situations where most of the existing mechanical foundation models are not applicable.

2. The development of horizontal stresses in the granular fill in the geosynthetic-reinforced granular fill - soft soil system resulted in settlement reductions for the set of parameters studied.

3. Prestressing the geosynthetic reinforcement in the geosynthetic -reinforced granular fill - soft soil system is found to be very effective in reducing both the total and differential settlements of the loaded region.

4. The compressibility of the granular fill has an appreciable influence on the settlement response of the geosynthetic-reinforced granular fill - soft soil system as long as the stiffness of the granular fill is less than approximately 50 times that of the soft soil.

5. There are several parameters e.g., interfacial friction coefficients, width of reinforced zone, prestress in the geosynthetic reinforcement, level of compaction of the granular fill etc. which may not govern the settlement behaviour of geosynthetic-reinforced granular fill - soft soil system significantly during initial stages of consolidation of the soft foundation soil, in spite of their higher values considered. However, in latter stages of consolidation process, these parameters, generally, show their beneficial effects in settlement reduction of the loaded region.

6. The numerical approach adopted is found to be very efficient in terms of economy of computations and takes only few seconds of CPU time to obtain the settlement and the mobilised tensile force in the geosynthetic reinforcement.

RECOMMENDATIONS FOR FURTHER WORK

1. Extension of the proposed foundation model to incorporate

i) nonlinearity of fill and soft subgrade

i) plasticity of fill and soft subgrade

iii) variation of K with depth of the granular fill

iv) variation of modulus of subgrade reaction with depth of the foundation soil and also with time

v) relative slip at fill-geosynthetic interface

vi) a generalised friction law, and

vii) creep and viscoelasticity of the geosynthetic reinforcement

2. Validation of the obtained results through laboratory and field model tests.

3. Improvement in solution technique.

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